"A STUDY OF ANISOTROPIC PERMEABILITY"

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BY

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CERTIFICATE

Certified that this work on "A STUDY OF ANISOTROPIC PERMEABILITY" has been carried out under my supervision and that this has not been submitted elsewhere for a degree.

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NOTATIONS USED

The following symbols have been adopted for use in the thesis.

a = Cross sectional area of the variable head tube unless otherwise specified.

A = Cross sectional area of the sample unless otherwise specified.

C = Shape constant

$d_1, d_2 \ldots dn$ = thickness of different soil layers

D_s = effective particle diameter.

e = void ratio

e_o = void ratio at the surface

e_{max} = Maximum Void ratio

e_{min} = Minimum Void ratio

e_{av} = Average Void ratio over a depth H.

G = Specific gravity of soil

h = depth from the surface unless otherwise specified

h_o = Initial head in the variable head tube.

h_1 = Final head in the variable head tube.

H = Constant head, unless other wise specified.

i = Hydraulic gradient.

K = Isotropic permeability of soil

K_{20} = Isotropic permeability at $20^{\circ}c$

K_T = Isotropic permeability at $T^{\circ}c$

$K_1, K_2 \ldots K_n$ = Isotropic permeability of different layers.

K_r = Radial permeability

K_x and K_H = Horizontal permeability

h

K_Z and K_v = Vertical permeability.

L = Length of the sample

n = porosity of the sample

$P, P_1 \& P_2$ = Force acting on the sample

Q = Quantity of water

Q_{cir} = Quantity of water flowing through a circular pipe.

R = Radius of the soil sample

R_H = Hydraulic radius

r_o = Radius of the porous rod

γ_K = Permeability ratio

γ_{Kmax} = Maximum permeability ratio

T = Temperature

t = time

V = Total volume of the sample

V_v = Volume of void

V_s = Volume of solid

W_s = Wt. of the solid

α = A constant

γ_ω = unit wt. of water

γ_{20} = unit wt. of water at $20^\circ c$

γ_T = unit wt. of water at $T^\circ c$

μ = Viscosity of water

μ_{20} = Viscosity of water at $20^\circ c$

μ_T = Viscosity of water at $T^\circ c$

η = Degree of parallelism.

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ABSTRACT

A radial permeameter unit is designed and developed. The instrument can be used for measuring radial permeability by constant as well as variable head methods. Appropriate expressions have been derived for the coefficient of radial permeability for both constant and variable head.

Factors on which permeability depends, different theoretical, semiempirical and empirical approaches to permeability and various field and laboratory methods of determining permeability coefficient, are presented & reviewed.

A study of radial and vertical permeability of kaolinite and the influence of the structure in the micro-level or degree of parallelism on these coefficients of radial and vertical permeability are made. Results are presented and discussed. (Evaluation of the data shows that flaky shape of clay particles and its orientation with reference to vertical and horizontal axes gives more permeability in the horizontal than vertical direction.)

A hypothesis for structural scale based on permeability ratio is presented and discussed.

A detailed investigation of 'shape factor' for Kalpi and Ganges sand, is made . Dependency of this shape factor on grain size, shape and void ratio is stressed. Results are presented and discussed.

An analytical solution for depth dependent anisotropic permeability for a linear decrease of void ratio is forwarded. The final equations relate the average void ratio over a depth to horizontal and vertical permeability and the permeability ratio.

CHAPTER 1

INTRODUCTION

Horizontal permeability is an essential parameter in seepag studies relating to design of vertical sand drains, earthdams, dewa ering systems etc. It is also an important criterion for two or thi dimensional consolidation problems. Increased permeability in the horizontal direction due to ani-sotropy of sedimented soil necessi- tate the measurement of permeability in the direction of bedding. The flat shape of clay particles and its orientation with respect to vertical axis in the homogeneous soil usually give rise to higher permeability in the horizontal direction.

Although there are several field techniques available (1,2 for estimating horizontal or radial permeability, (k_h or k_r) there is no direct method of finding it in the laboratory. Taking a hori- zontal core sample from the field and running a vertical permeabil tes was the only laboratory technique available uptill now. This method for the laboratory determination of horizontal permeability of soil is cumbersome and subject to errors introduced in obtain- ing a specimen with planes of bedding truely vertical.

The instrument developed here, enables the determination of the radial permeability characteristics under controlled condi- tions in the laboratory. By preparing two idential samples and

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*Numbers in the brackets refer the reference number, the list of which is given at the end.

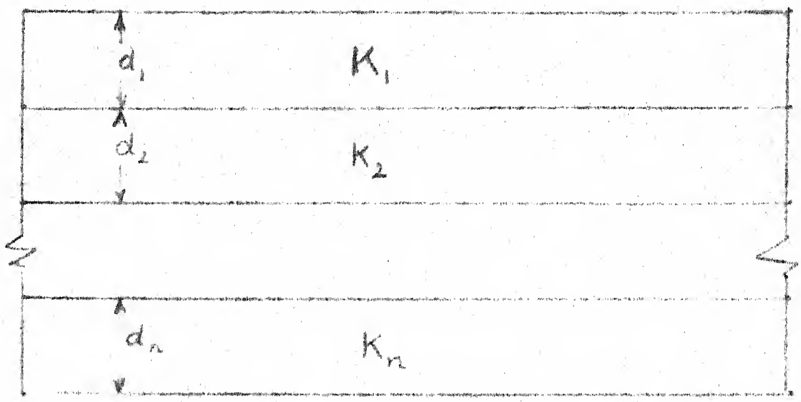
running one test in the radial permeameter and the other in the vertical permeameter under identical conditions, it is possible to study the effect of different variables on radial and vertical permeabilities and their interrelation-ship between the two, if any.

In the usual design of earthdams and other earth structures it is generally assumed that horizontal permeability is equal to the vertical permeability. This assumption is of doubtful validity and is likely to be on the unsafe side, particularly in case of clay whose directional permeability is highly sensitive to its structure and its particle orientation or degree of parallalism. The experimental results presented here also proves this fact. Arbitrary values of ratios of horizontal to vertical permeabilities have also been suggested (38) which appears too conservative. In such cases a rational basis for estimating the horizontal permeability k_h, is suggested. A theoretical relation between degree of parallelism, void ratio and permeability ratio (k_r/k_v) based on some simplifying assumptions as supported by experimental results is also presented. An hypothesis for structural scale for soil as a function of permeability ratio γ_k and void ratio is presented.

Evans in 1962 (4) have shown analytically that the permeability in the horizontal direction is always greater than the permeability in the vertical direction for a stratified soil, as shown below.

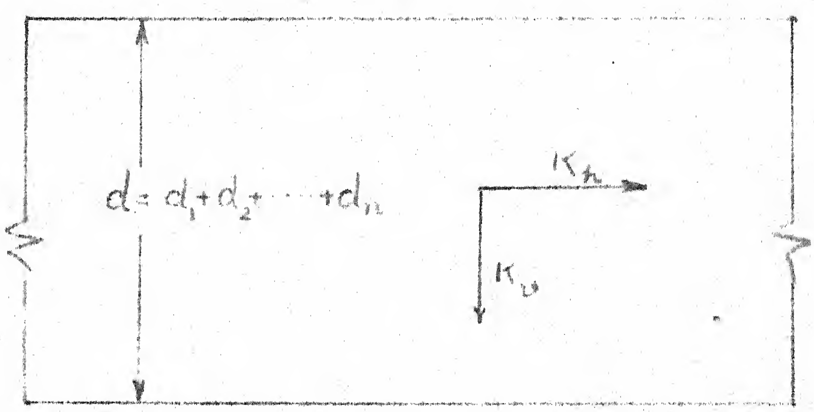
With reference to fig. 1 (a) and (b) and with the help of Darcy's law, we have

FIG. 1(a)



'n' LAYER SOIL SYSTEM

FIG. 1(b)



EQUIVALENT SOIL SYSTEM

$$k_h = \frac{k_1 d_1 + k_2 d_2 + \dots\dots + k_n d_n}{d} \qquad \dots\dots\dots\dots\dots \quad 1.1$$

$$k_v = \frac{d}{\frac{d_1}{k_1} + \frac{d_2}{k_2} + \dots\dots + \frac{d_n}{k_n}} \qquad \dots\dots\dots\dots\dots \quad 1.2$$

Therefore by eqn 1.1 & 1.2

$$\frac{k_h}{k_v} = (k_1 d_1 + k_2 d_2 + \dots\dots + k_n d_n)\left(\frac{d_1}{k_1} + \frac{d_2}{k_2} + \dots + \frac{dn}{k_n}\right) \times \frac{1}{d^2}$$

$$= \left(\frac{d_1}{d}\right)^2 + \left(\frac{d_2}{d}\right)^2 + \dots + \left(\frac{d_n}{d}\right)^2 + \frac{d_1 d_2}{d^2}\left(\frac{k_1}{k_2} + \frac{k_2}{k_1}\right) + \frac{d_2 d_3}{d^2}\left(\frac{k_2}{k_3} + \frac{k_3}{k_2}\right)$$

$$+ \dots \frac{d_{n-1} d_n}{d^2}\left(\frac{k_{n-1}}{k_n} + \frac{k_n}{k_{n-1}}\right)$$

or $$\frac{k_h}{k_v} = \left(\frac{d_1}{d}\right)^2 + \left(\frac{d_2}{d}\right)^2 + \dots + \left(\frac{d_n}{d}\right)^2 + \frac{d_1}{d}\frac{d_2}{d}\left(\frac{k_1}{k_2} + \frac{k_2}{k_1}\right) + \frac{d_2}{d}\frac{d_3}{d}\left(\frac{k_2}{k_3} + \frac{k_3}{k_2}\right)$$

$$+ \dots\dots + \frac{d_{n-1}}{d}\frac{d_n}{d}\left(\frac{k_{n-1}}{k_n} + \frac{k_n}{k_{n-1}}\right)$$

Let $\frac{d_1}{d} = a_1$ $\quad \frac{d_2}{d} = a_2$ $\quad$ etc.

Therefore the above equation becomes

$$\frac{k_h}{k_v} = (a_1^2 + a_2^2 + \dots a_n^2) + a_1 a_2\left(\frac{k_1}{k_2} + \frac{k_2}{k_1}\right) + a_2 a_3\left(\frac{k_2}{k_3} + \frac{k_3}{k_2}\right)$$

$$+ \dots - a_{n-1}\, a_n\left(\frac{k_{n-1}}{k_n} + \frac{k_n}{k_{n-1}}\right)$$

or $$\frac{k_h}{k_v} = \sum a^2 + \sum a_p\, a_q\left(\frac{k_p}{k_q} + \frac{k_q}{k_p}\right) \qquad \dots\dots\dots\dots\dots \quad 1.3$$

Let $\frac{k_p}{k_q} = m \quad \therefore \quad \frac{k_q}{k_p} = \frac{1}{m}$

$$\therefore \frac{k_p}{k_q} + \frac{k_q}{k_p} = m + \frac{1}{m} \quad \ldots\ldots\ldots\ldots\ldots \quad 1.4$$

$$\therefore \frac{d}{dm}\left(\frac{k_p}{k_q} + \frac{k_q}{k_p}\right) = 1 - \frac{1}{m^2} \quad \ldots\ldots\ldots\ldots\ldots \quad 1.5$$

$$\& \quad \frac{d^2}{dm^2}\left(\frac{k_p}{k_q} + \frac{k_q}{k_p}\right) = 0+2/m^3 \quad = \text{positive}$$

to get minimum value of $\left(\frac{k_p}{k_q} + \frac{k_q}{k_p}\right)$, from equation 1.5

$$\frac{d}{dm}\left(\frac{k_p}{k_q} + \frac{k_q}{k_p}\right) = 1 - \frac{1}{m^2} = 0$$

$$\therefore m = 1$$

From equation 1.4

$\left(\frac{k_p}{k_q} + \frac{k_q}{k_p}\right) \geqslant 2$ for all positive value of k_p and k_q

Therefore from equation 1.3

$$\frac{k_h}{k_v} \geqslant \sum a^2 + \sum 2a_p a_q$$

$$\text{or} \quad \frac{k_h}{k_v} \geqslant \sum (a)^2$$

$$\text{or} \quad \frac{k_h}{k_v} \geqslant (a_1 + a_2 + \ldots + a_n)^2$$

$$\text{or} \quad \frac{k_h}{k_v} \geqslant \left[\left(\frac{d_1}{d}\right) + \left(\frac{d_2}{d}\right) + \ldots\ldots + \left(\frac{d_n}{d}\right)\right]^2$$

$$\text{or} \quad \frac{k_h}{k_v} \geqslant \left[\frac{d_1 + d_2 + d_3 + \ldots + d_n}{d}\right]^2$$

$$\text{pr} \quad \frac{k_h}{k_v} \geqslant \left[\frac{d}{d}\right]^2 \quad \text{or} \quad \frac{k_h}{k_v} \geqslant 1$$

$\frac{k_h}{k_v}$ equals to 1 when $k_1 = k_2 = k_3 = \ldots k_n$ i,e when the whole system is hydraulically homogeneous.

Hence for layered system when $k_1 \neq k_2 \neq \ldots k_n$

$$k_h > k_v \quad \text{or} \quad \gamma_k > 1.$$

The above proof, as was originally given by Evans, clearly proves the greater horizontal permeability in a layered soil but even in a single layered soil where the soil may be compositionally or otherwise (e,g, grain size, shape etc.) homogeneous may show aniso-sotropic hydraulic behaviour due to the change of void ratio with depth. The void ratios of soils depend upon the consolidation pressure and hence it is expected that within a given formation the void ratios generally decrease with depth which give rise to a well known phenomena known as depth dependent ani-sotropic permeability. Assuming a linear decrease of void ratio with depth a complete analytical solution for this depth dependent ani-sotropic permeability is also presented here.

For an one layer homogeneous clay stratum (when there is no void ratio or density variation along the depth) the presence of difference between k_h and k_v due to particle shape and its orientation with respect to horizontal and vertical axes have not been proved mathematically uptill now. The logic, reasoning and experimental evidences only can justify this phenomena. This present investigation also tried to explain qualitatively as well as quantitavely the same phenomema.

For granular soils, the permeability coefficiant can be satisfactorily found out from the following equation as originally developed by Taylor (3), subject to the proper

$$k = D_s^2 \frac{\gamma}{\mu} \frac{e^3}{1+e} C \quad \ldots\ldots\ldots\ldots\ldots \quad 1.6$$

evaluation of the shape factor 'C' and effective particle diameter 'D_s'. An experimental study of this shape factor utilising the above equation 1.6 is made and also presented.

CHAPTER 2

A GENERAL DISCUSSION AND REVIEW OF LITERATURE

2.1 DARCY'S LAW AND PERMEABILITY COEFFICIENT:

The facility with which water is able to travel through the soil pares has much significance in many types of engineering problem. This property of soil is commonly called as permeability or hydraulic conductivity. The classic equation of water movement set forth by Henry Darcy (11) in 1856 occupied a unique place in the study of fluid flow through porous media. The Darcy flow equation

$$v = ki \quad \ldots\ldots\ldots\ldots\ldots \quad 2.1$$

expresses the proportionality between the superficial flow velocity (v) and the driving force expressed in terms of hydraulic gradient (i). The 'k' in this equation, the Darcy k, is commonly used by soil scientists and engineers as a practical unit for expressing the permeability of soil to a fluid. Regarding the nomenclature of this constant 'k' there was some controversy and some people used to call it as permeability coefficient and others preffered to call it as hydraulic conductivity. So to avoid this confusion soil Science Society of America formed a committee on the terminology issue (12) which published : its report in 1952 and which gave the following recommendation.

"Since the Darcy equation is directly and completely analogous to the Fourier law for the flow of heat in which case the proportionality factor is called the "Thermal conductivity"; and is also analogous to Ohm's law for the flow of electricity in metals in which case the proportionality factor is referred to as the "electrical

conductivity". Some such term as "Water conductivity" should be suitable for the Darcy 'k' in connection with the flow of water in soil. "Hydraulic conductivity" would be a broader term suitable for use in connection with saturated flow of any specified liquid.

Now coming down to the limitations of Darcy's law, Darcy himself recognised that his relationship was not valid for high fluid velocities. During past 40 years, much research has been concerned with the nature of this deviation which occures at large hydraulic gradients (13,14). It seems well established that when the hydraulic gradient exceeds a critical value, the flow velocity is no longer proportional to the hydraulic gradient, but increases less rapidly than gradient (15).

Compared with the extensive study conducted on deviations at large gradients, much less work has been directed towards the testing of Darcy's law for fluid at low gradients, even in soils, where such flow is common. It has been suggested by Florin (16) that, as a result of physicochemical interactions between the soil and water in clays, seepage will not occur until a certain limiting gradient, called threshhold gradient 'i_o' is surpassed. Langfelder, Chen and Justice (18) also observed the same phenomena. Hubbert (17), on the other hand indicated that there was no apparent reason to suspect the validity of the Darcy's law at low gradients. Scheidegger (39) however, has also recognised the possibility of deviations

arising from a so called "boundary effect" from ions in solution, and from non Newtonian fluids. Swartzendruber (15) has proposed a modified equation for liquid flow in porous media containing clay, for which the flow is considered to be non Newtonian. The equation has two parameters, and includes Darcy's law as a special case.

The pores of most soils are so small that flow of water through them is laminar. However in very coarse soils the flow may sometimes be turbulent. Fancher, Lewis and Barnes have established a criterian that a Reynolds no. of 1 corresponds to the begining of turbulant flow in porous media but hawever, Anandakrishnan and Varadajalu (13) have found that the flow was turbulent for the sand with effective grain size d_{10}= 0.3 mm, even though the Reynold no. was substantially less than 1. Anandakrishnan and Varadarajulu again in the same paper (13) have proposed an equation of flow through the sand under conditions of turbulance. The equation was of the form

$$v^n = k'i \qquad \text{, ...2.2}$$

where v = velocity of flow

i = Hydraulic gradient

k' = a constant called coefficient of turbulent flow

n = Another constant, turbulance exponent as the authors called.

The authors have assumed that voids in soils form a system of continuous pipes for which the above equation proposed is

analytically true.

2.2 THEORETICAL, EMPERICAL AND SEMIEMPIRICAL APPROACHES FOR PERMEABILITY:

Several attempts have been made to get a theoretical equation for permeability coefficient. Out of these the most notable one is that of Kozeny and Carman (19,20) and the final equation of whose reads as

$$k = \frac{1}{k_o s^2} \quad \frac{\gamma}{\mu} \quad \frac{e^3}{1+e} \quad \ldots\ldots\ldots \quad 2.3$$

where k_o = constant depending on pore shape and ratio of length of actual flow path to soil bed thickness

and s = specific surface area of the soil particles.

From a comparison of flow through soils with flow through capillary tubes, Taylor (3) has developed the following equation

$$k = D_s^2 \frac{\gamma}{\mu} \frac{e^3}{1+e} C \quad \ldots\ldots\ldots\ldots \quad 2.4$$

where C = is a shape factor & D_s some effective particle diameter.

As 'D_s' is defined as the diameter of particle having specific surface s, equation 2.4 may be considered a simplification or extension of Kozeny Carman equation.

So far as emperical and semiempirical approaches are concerned, starting from Hazen (1892) to uptill now numerous attempts have been made to evolve an easy, compact and handy equation for permeability from the laboratory and available field

test datas. A brief summary of all these empirical and semiempirical formulae are nicely presented by Landon (21). From the permeability test results of filter sands of grain size 0.1 to 0.3 mm of fairly uniform grain size (U = 5), Hazen in 1892 suggested a simple equation

$$k = C\, d_{10}^2 \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad 2.5$$

where C = a constant, the value of which may vary from 41 to 146. Slichter in 1899 derived the following formula for uniform spheres of diameter 'd'. The value

$$k = \frac{771\, d^2}{C} \quad \ldots\ldots\ldots\ldots\ldots \quad 2.6$$

of the shape constant 'C' was derived by him for different pore geometries and tabulated them as follows

Porosity n	0.26	0.28	0.30	0.34	0.38	0.42	0.46
C	84.3	65.9	52.5	34.7	24.1	17.3	12.8

Following the procedure given by Slichter but extending it to cover sand of non uniform grain size and variable grain shape Tarzaghi in 1925 proposed another equation which reads as

$$k = \frac{C}{\mu}\left[\frac{n - 0.13}{\sqrt[3]{1-n}}\right]^2 d_{10}^2 \quad \ldots\ldots\ldots\ldots \quad 2.7$$

where C = is shape constant as usual

and μ = viscosity of water

n = porosity

The above equation includes an empirical term (n-0.13). The parameter $\frac{C}{\mu}$ varies from 800 for rounded sands to 460 for angular sands.

Kozeny in 1927 developed another expression for permeability following an extension of poiseulle's equation for the flow through capillary tubes. His equation looks like as follows

$$k = \frac{g}{k' \mu s^2} \left[\frac{n^3}{1-n^2} \right] \quad \ldots\ldots\ldots\ldots \quad 2.8$$

where g = Accleration due to gravity

k' = a constant equal to 5 for spherical grains

s = Specific surface of angular material

μ = viscosity of water

n = porosity

This Kozeny's equation was verified experimentaly by Carman in 1938.

Rose in 1950 proposed another formula as below.

$$k = \frac{gd^2}{1000\,\mu} \frac{1}{f(n)} \quad \ldots\ldots\ldots\ldots \quad 2.9$$

where $f(n)$ = is a function relating to 'relative resistance' to porosity, equal to unity at $n = 40\%$

All the equations discussed above have been found to express satisfactorily the permeability characteristics of saturated sand subject to the proper evaluations of shape constants and other parameters. On the otherhand, laboratory testing definitely shows that all these equations are far from correct for clays. The probable reasons for this disagrement for fine grained soils as suggested by Taylor (3) is

"A thin surface film of water, which is bound to all particles, and water, which is bound between

parallel, plate shaped soil particules, are the probable explanations. Because of this bound water seepage occurs only through a part of pore space. Possibly these equations may be correct under a revised concept wherein the void ratio is replaced by the ratio between the volume of free water and all other volume. However, no method is available at present for obtaining values of this ratio".

Lambe (5) had put the reasons of disagreement in the following words

"The permeability equations are of very limited use to the soil engineer for finegrained soils for two reasons: (1) the difficulty of selecting the effective 'constants' and soil characteristics, and (2) the fact that these various terms are not independent, but interrelated in a very complex manner. One can well argue for discarding the equations when considering fine grained soils; one can also argue that the equations are sound but that the knowledge of soils is not extensive enough to permit proper interpretations of the equations".

Whatever may be the reasons of disagreement, it has been found, experimentally, that a plot of the void ratio to natural scale against the coefficient of permeability to logarithmic scale approximates a straight line for any fine grained clayey soil.

2.3 PERMEABILITY MEASUREMENTS IN THE LABORATORY AND FIELD

In laboratory usually, two types of permeameters are used to determine the vertical permeability and they are (a) constant head vertical permeameter and (b) variable head vertical permeameter, the description and underlying principal of which are fairly easy and well known and is available in any standard text book (3). A radial permeameter both constant head type and variable head type is being developed as a part of author's research project and is reported here in.

Regarding field methods of determining permeability, Don Kirkham (1) reported an exhaustive survey of the available field methods and tried to pick up the relative advantages and disadvantages of each method. The methods described by him are

(a) Augerhole method

(b) Piezometer method

(c) Tube method

(d) Childs two well method

(e) Dry auger hole method

(f) Four well method

(g) Single well method

The underlying principles of all these methods are same but techniques and practical details varied. In all these methods, the time for the water to rise a certain distance in the cavity or hole or in well, as the case may be, is observed and this

time and distance are used finally in a suitable formula to yield permeability af soil in place.

Stallman & Smith (22) also reviewed and proposed some field methods of permeability in relation to ground water investigation. They have devised a special type of sampler of the piston type containing an inner barrel, in which an undisturbed soil sample is taken. This inner barrel, with its undisturbed sample, is removable and serves as the permeameter tube in subsequent permeability measurements.

2.4 PERMEABILITY IN THE ANISOTROPIC SOIL

Distinct stratification in the natural soil brings ani-sotropy and permeability of this type of soil is different in any two mutually perpendicular directions. The change in density along the depth and the flaky shape of clay particles also may cause ani-sotropic permeability which is discussed in detail in subsequent chapters.

Permeability in the aniosotropic soils were fully covered by Childs, Colligeorge & Holmes (23) and also by Child alone (24) and again by Talsma (2). All of these engineers and scientists tried to separate horizontal and vertical component (k_h&k_v) of permeability in field and have come out with a solution which are more or less alike. Flannery and Kirkham (40) proposed an apparatus for determination of insitu horizontal permeability. Mansur and Dietrich (25) reported the results of several pumping tests in the alluvial valley of Arkansas river with a view to get horizontal permeability (k_h) and permeability ratio ($\frac{k_h}{k_v}$). The average value of the permeability

ratio for the site was found to be 2. Various sand strata of the alluvial valley of the Mississippi river was also tested for horizontal permeability by Pumping test by Mansur (26). Results of all these investigations showed that in all cases (except in some special cases when deep root channels, vertical fissures etc. are there) horizontal permeability is more than the vertical, the mathematical proof of which was forwarded by Evans (4) and is shown is chapter 1. Intuitively also this seems to be correct because it can be imagined that the fine grained laminae, separating coarser grained laminae, would directly hinder vertical water movement through the deposit, whereas they could not be equally effective in hindering horizontal water movement. A short review of the past work in depth dependent anisotropic permeability is given in chapter 8.

2.5 EFFECT OF SOIL COMPOSITION AND PORE FLUID ON PERMEABILITY

The influence of soil composition on permeability is generally of little importance with silts, sands and gravels but with clays, it plays a very important part. Cornell University, in a report (27), reported some test results for permeabilities of the various ionic forms of montmorillolite which shows that in general, the permeabilities of different montmorillonites are in the order of Ca > H > Na > K. Na-montmorillonites has permeability of less than 10^{-7} cm/sec. even at a void ratio of as high as 15 and that is one of the reason why Na-montmorillonite is widely used by the engineers as an impermeabilizing additive to other soils. The report also revealed that at a void ratio of 7, the permeability of Na-Montmorillonite is as high as 300 times the permeability of K-montmorillonite.

Michaels and Lin (28) studied the permeability of saturated Kaolinite to various fluids. As seen from equation 2.4 the value of absolute permeability ($k_{abs} = \frac{k\mu}{\gamma}$) should remain same for all fluid at the same void ratios. But the experimental datas published by Michael and Lin and also by others show that it is not true indicating thereby that there must be some other properties of fluid other than μ and γ which influences the permeability value. Fluid polarity which influences electroosmotic backflow and thickness of adsorbed water, may be another important property affecting permeability in fine grained soils (5). For granular soils, ofcourse, viscosity and density of fluid are perhaps the only properties influencing the permeability. Muskat (31) gave a comparison of absolute permeability values determined from flow of water and from the flow of air through the voids of large number of soils ending in a disaggrement between the values. He forwarded, change of structure due to the removal of water from the voids, as a possible reason for this.

2.6 EFFECT OF STRUCTURE AND TEMPERATURE ON PERMEABILITY

Permeability in fine grained soils depends to a considerable extent on the arrangement of soil particles or "structure". The importance of structure on almost all soil properties has been recognised, and Lambe (29) have forwarded a theoretical explanation for it. Regarding the evaluation of the 'structure' term, Lambe in another article (5) have said,

> "To evaluate directly a 'structure' term for the permeability equations will be exceedingly difficult. Attempts to measure the extent of aggregation have been made (30), but no simple way of giving soil a number to

indicate accurately its position in structure scale has been developed. The concept of scale ranging from 0 for complete dispersion to 100 for complete aggregation is emphasized to some extent at present, even though the best method of determining such numbers has yet to be established. The permeability of fine grained soils varies as some power of this "structure coefficient".

Taking the clue from this valuable comment an attempt has been made to evolve a structural scale for soils and is reported in chapter 6.

In considering the effect of temperature on the permeability of soils, it is necessary that we first consider the pertinent variables. The variables are: (a) Size, shape and uniformity of the particles; (b) Void ratio; (c) Presence and physical properties of the adsorbed films; (d) Viscosity and unit weight of free liquid. A review of these variables indicates that temperature will affect the viscosity and unit weight of the liquid and the adsorbed film, if it is present. Since the viscosity of water decreases with increasing temperature permeability should increase with the increase of temperature. A very simple equation has been proposed (32) to express the relationship between temperature and permeability, based on the variation of viscosity and unit-weight with temperature. The equation is

$$k_2 = \frac{\mu_1}{\mu_2} \, \frac{\gamma_2}{\gamma_1} \, k_1 \quad \ldots\ldots\ldots\ldots \quad 2.10$$

where the subscripts refer to two different temperatures Buchanan (33) have studied exhaustively the effect of temperature on permeability and finally have came to the conclusion that, temperature affects the viscosity of the free liquid and the adsorbed film, but the effect, in temperature range of concern (45°F to 105°F), is small.

2.7 ROLE OF ION EXCHANGE AND SWELLING CHARACTERISTICS ON PERMEABILITY

When clays are present in a sediment, many complications arise in the permeability measurement. Ordinary gases, for example, may be adsorbed by the clay complex. If water is used as the permeant, the physical properties of the clay are altered, according to the chemical composition of water. If the water is distilled to a high degree purity and then passed through a calcium saturated montmorillnite clay the calcium will be leached out and a hydrogen saturated montmorillonite formed by an exchange of ions and as pointed out earlier, the permeability of this hydrogen saturated montmorillonite will be entirely different from that of calcium montmorillonite. Again, montmorillonite has several times the ion exchange capacity than kaolinite and therefore is capable of greater changes in physical properties because of ion exchange.

The above considerations seem to indicate that in permeability measurements, it is necessary to use water which is similar in composition to that which occurs in, or ultimately be passed through the sediment. This point was first emphasized by Smith and Stallman (22). They exemplified it by saying, "permeability of the sediments lining

an irrigation ditch should be measured with water typical of that used for the irrigation. Permeabilities of sediments below the water table should be measured with ground water taken from those sedimen

Swelling characteristics of soils has an important bearing on its hydraulic properties. Miss Foster (34) has thrown much light on this aspect. She reports that significance of swelling as a factor affecting permeability measurements of clay cantaining sediments depend on the kind of clay present. If the clay is kaoline or hydrous mica the effect of swelling would be of little importance as these clays swell ttle, but it is of great importance when the clay present is sodium montmorillonite because this swells maximum. Differences in the swelling characteristics of different clays may be related to their crystal structure, chemical composition and to the amount and nature of their associated exchangable cations.

2.8 EFFECT OF AIRBUBBLES OR DEGREE OF SATURATION ON PERMEABILITY

When the soil is incompletely saturated, the coefficient of permeability will be smaller than when saturation is complete. Polubornova and Kochina (14) have found out experimentally that the ratio of permeability of the unsaturated soil to that of the saturated material at the same void ratio varies approximately as the degree of saturation (S/100) to the power 3.5 over the range of saturation from zero to 100%. However, in the range of degree of saturation which is of most interest in soil engineering, that is, from 80 to 100% the ratio of the permeabilities above is nearly a linear function of the degree of saturation and varies as $\left[1-m(1-S/100)\right]$, where 'm' is a

constant with values between 2 & 4. The Linear approximation to the power curve in the 80 to 100% range has an 'm' value of 3.5. Orlob and Radhakrishnan (35) found indication that the lower values of 'm' hold for soils of uniform grain size and that 'm' increases in well graded materials. Pillsbury and Appleman (36) observed that

(i) the maximum effect of trapped air apears to be in pores of intermediate size.

(ii) this trapped air is removed only by solution with water percolating through the soil. The ease with which air is dissolved depends on the capacity of water to absorb air and on the time of contact of that water with air, and more important, with the amount of percolating water passing through per unit amount of trapped air.

Anyway, uptill now sufficient datas are not available to co-relate degree of saturation with the permeability with a good amount of confidence.

2.9 ROLE OF MICRO-ORGANISM IN PERMEABILITY

Allison (37) first observed that a soil which is under prolonged submergence shows a definite decrease of permeability with time. He justified and experimentally proved that their decrease is due to

1. Biological clogging of soil pores with microbial cells and their synthesised products (slimes or polysaccharides)

ii. A dispersion due to attack of micro-organisms or organic materials which bind soils into aggregates.

Allison conducted two series of tests, the 1st series with distilled water and the soil was treated . ii with ethylene oxide to get rid of bacteria and which do not alter the physical and chemical properties of soil and the 2nd series of tests without any sterilisation, keeping all other things constant. The 1st series did not show any decrease of permeability value with time whereas the 2nd series showed a steady and gradual decrease of permeability with time which undoubtedly supports the fact that for a soil which is under prolonged submergence, micro-organisms do take a part in reducing the permeability value of the soil.

2.10 CONCLUSION

Permeability characteristics have been a subject of investigation by many hydrologists, engineers and physicists since the times of Darcy and Hagen, a centary ago. But from the above short review, it might be clear that there exists still problems to be solved, particularly those relating to the effect of clay minerals. A more vigorous and exhaustive research is necessary to unravel the mystries and delicacies of permeability characteristics of soils.

CHAPTER 3

RADIAL PERMEAMETER

3.1 DESIGN AND DESCRIPTION OF RADIAL PERMEAMETER:

A radial permeameter unit has to satisfy the following two conditions:

(a) The flow through the soil specimen should be in true radial direction.

(b) It should be capable of being used as a constant head or variable head permeameter.

To meet these basic requirements a unit as shown in fig.2 was fabricated. The equipment consists of a porous cylinder 10 cm in internal diameter and 10 cm long which serves as the container for the soil specimen. The cylinder is enclosed between two steel plates with rubber seals between the cylinder and the steel plates. The top and bottom plates are tightened by means of four bolts. A porous rod 1.3 cm. diameter is positioned centrally within the soil sample. The bottom end of the porous rod is connected to a rubber or plastic outlet tube through a central hole in the bottom plate. The space around the outlet tube in the bottom plate is sealed for water tightness. This assembly is placed on a tripod within a water-tight closed cast iron cylinder filled with water. The top plate of the container is provided with an inlet for connection to a constant or variable head. The bottom of the cast iron container is provided with a short tube, one end of which is connected to the outlet tube from the porous rod. The other end of the outlet tube is connected to

RADIAL PERMEAMETER

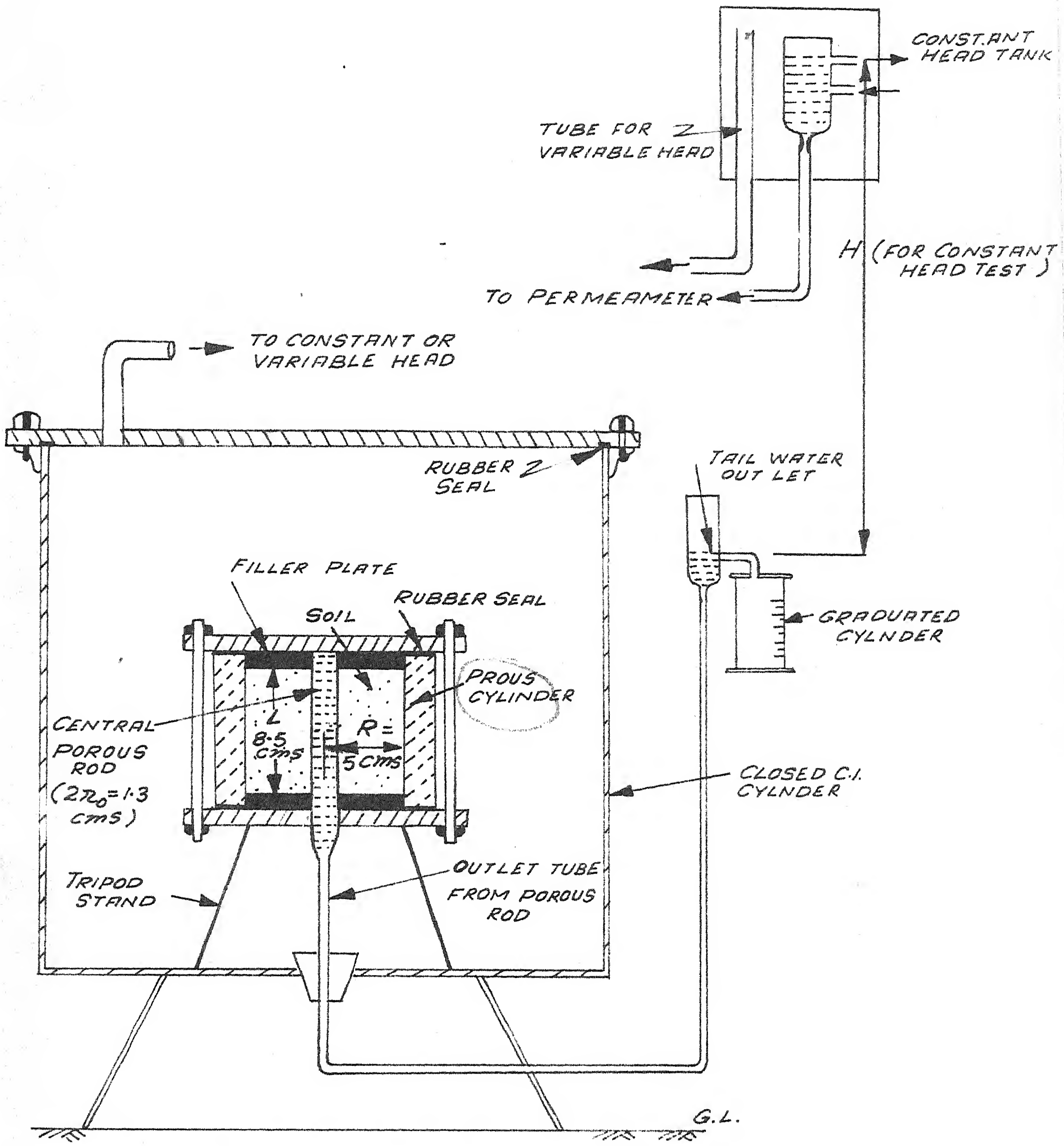


FIG. 1

a tailwater outlet kept at a level higher than the top level of the porous rod. Leakage through the periphery of the top cover of the outside cylinder was prevented by rubber seal all around.

For the central vertical drainage core, it would have been ideal to use a porous rod whose permeability is very large as compared to the soil. Porous rod of such quality was not available. Hence the central drainage core was made up of a brass tube 1.3 cm. dia. with a large number of perforations covered by a fine wire mesh and filter paper.

The whole radial permeameter unit was tested for leakage under 7' head of water for 24 hours and there was no leakage.

3.2 FLOW MECHANISM IN THE UNIT:

When the inlet pipe on the cast iron container is connected to a head, constant or variable, the corresponding hydrostatic pressure is induced in the water inside the container. The total head along the periphery of the cylinder consisting of pressure head plus elevation head remains constant at any instant of time. Similarly the total head along the periphery of the porous rod also remains constant. The difference in head between the heads on the outside and inside periphery of the soil specimen causes a truely radial flow. The fall in head through the walls of the cylinder is considered negligible.

The quantity of water flowing through the soil specimen is passed through the porous rod and the outlet tube arrangement which is collected at the tailwater outlet. The porous rod and the outlet tube are kept filled with water at the beginning of

the test.

3.3 DEVELOPMENT OF EXPRESSION FOR RADIAL PERMEABILITY COEFFICIENT (k_r) FOR CONSTANT HEAD:

Let Q, be the quantity of water in time 't'. Let 'R' be the inside radius of the porous cylinder and 'r_o' the radius of the porous rod. Le 'L' be the length of specimen (fig.3). Let H, be the constant head difference between the tailwater and head water levels. Taking tailwater level as the datum, the total head along the periphery of the porous cylinder over the entirelength is H. For the same datum the total head along the periphery of the porous rod is zero.

Let'h' be the head at a radial distance 'r' from the centre line of the porous rod. Then the quantity of flow

$$Q = k_r \frac{dh}{dr} 2\pi r L t$$

or $$Q \frac{dr}{r} = 2\pi L k_r dh$$

or $$Q \int_{r_o}^{R} \frac{dr}{r} = 2\pi L k_r t \int_{0}^{H} dh$$

or $$Q \log_e \frac{R}{r_o} = 2\pi L k_r t H$$

or $$k_r = \frac{\log_e \frac{R}{r_o}}{2\pi L H} \quad \frac{Q}{t} \quad \ldots\ldots\ldots\ldots \quad 3.1$$

Equation 3.1 gives the coefficient of radial permeability when the sample is subjected to a constant head H.

RADIAL FLOW UNDER CONSTANT HEAD

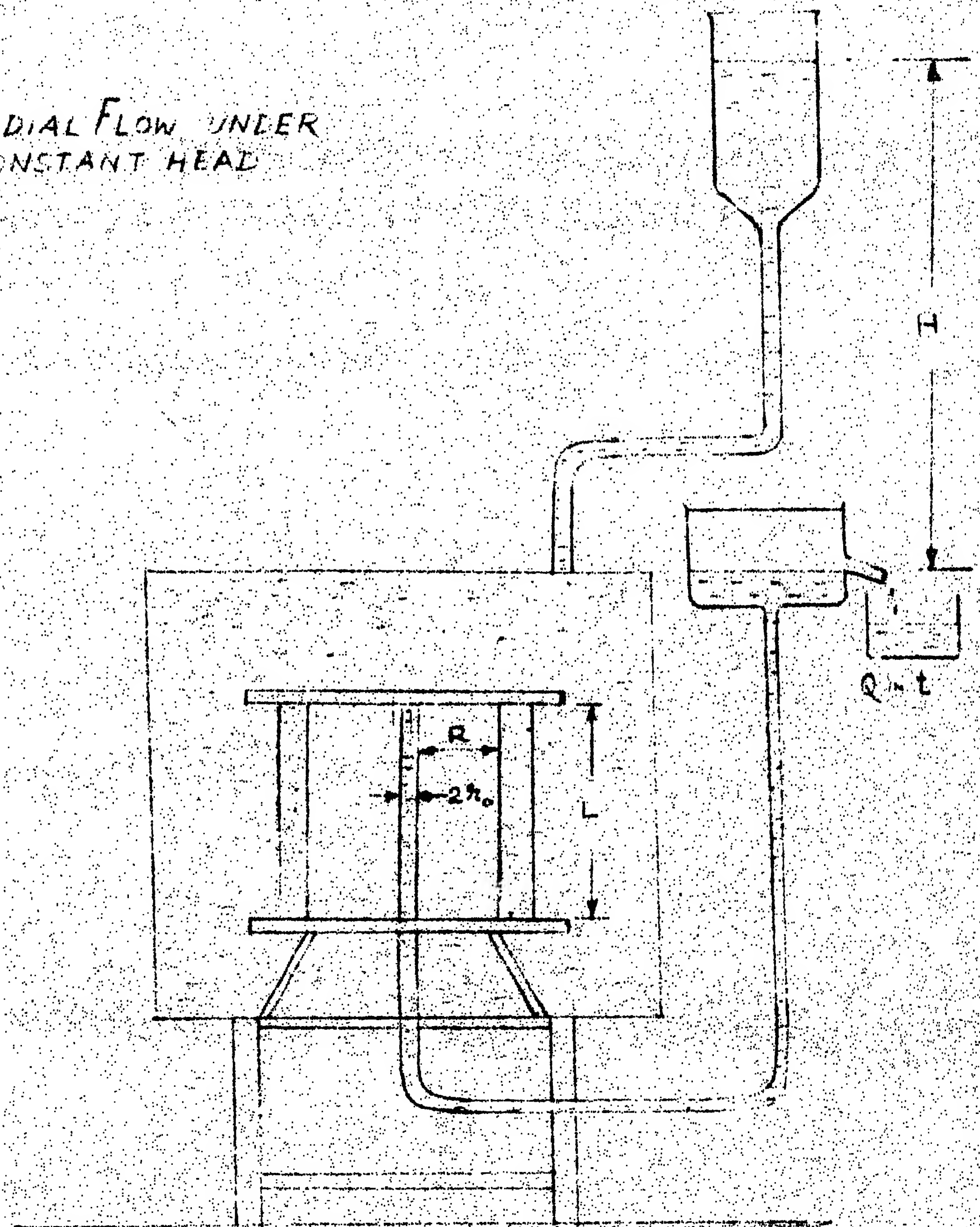


FIG. 3

3.4 DEVELOPMENT OF EXPRESSION FOR COEFFICIENT OF RADIAL PERMEABILITY (k_r) FOR VARIABLE HEAD:

Let 'h' be the variable (fig.4) head difference at any instant of time 't'. Taking tailwater level as the datum, the total head along the periphery of the porous cylinder at time 't' over its length remains 'h'. Let 'a' be the cross sectional area of the stand pipe. Let 'dh' be the change in head during an interval of time 'dt'. Using equation 3.1 the quantity of flow during this interval of time 'dt' is expressed as

$$q = \frac{2\pi L h k_r}{\log_e \frac{R}{r_o}},$$

where q = rate of outlet water $= \frac{Q}{t}$ = rate of inlet water $= -a\frac{dh}{dt}$

& h = Total head at the instant 't'

$$-a\frac{dh}{dt} = \frac{2\pi L h k_r}{\log_e \frac{R}{r_o}}$$

$$\text{or} \quad -\frac{dh}{h} = \frac{2\pi L k_r}{a \log_e \frac{R}{r_o}} dt$$

$$\text{or} \quad -\int_{h_o}^{h_1} \frac{dh}{h} = \frac{2\pi L k_r}{a \log_e \frac{R}{r_o}} \int_0^t dt$$

$$\text{or} \quad \log_e \frac{h_o}{h_1} = \frac{2\pi L k_r}{a \log_e \frac{R}{r_o}} t$$

$$\text{or} \quad k_r = \left[\frac{a \log_e \frac{R}{r_o}}{2\pi L}\right] \frac{\log_e \frac{h_o}{h_1}}{t} = \lambda_r \frac{\log_e \frac{h_o}{h_1}}{t} \quad \ldots \quad 3.2$$

where $\lambda_r = \frac{a \log_e \frac{R}{r_o}}{2\pi L}$.

RADIAL FLOW UNDER VARIABLE HEAD.

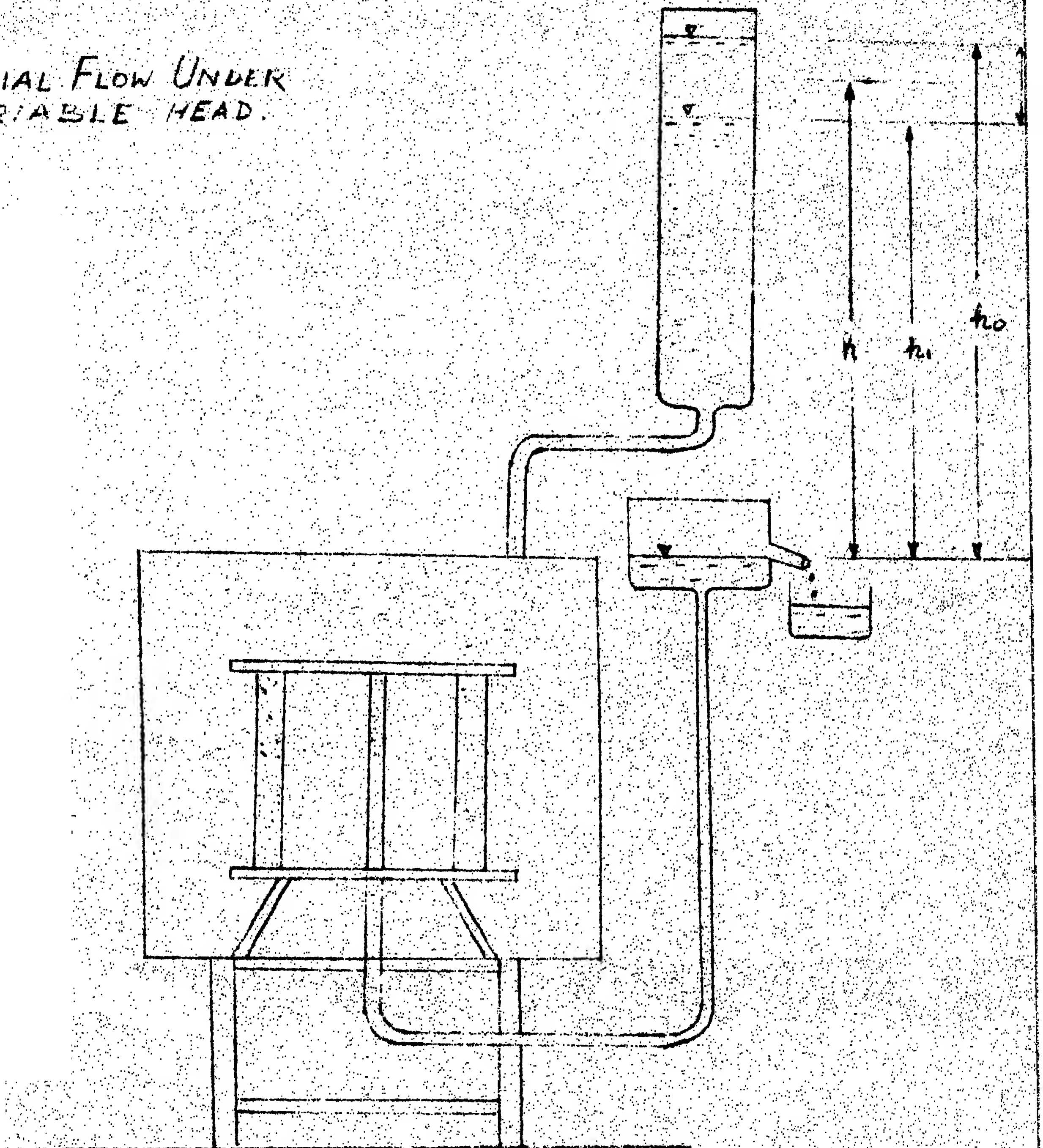


FIG. – 4

Equation (3.2) gives the coefficient of radial permeability for variable head test.

3.5 Experimental verification of the analytically derived expressions for coefficient of radial permeability for constant and variable head.

Using the radial permeameter, tests were conducted on a kaolinite soil to determine K_r by constant and variable head. Any soils should show exactly same coefficient of permeability for both variable head and constant head provided the expressions used to determine the same is analytically sound and correct. Test results for variable head is shown in table 1 below. And the results for the same sample a at the same two,

TABLE 1

RADIAL PERMEABILITY OF KAOLINITE (Variable head)

a = cross-sectional area of the variable head tube = 1.052 sq.cm.

R = Inside radius of the porous cylinder = 5 cm.

r_o = Radius of the porous central rod = 0.65 cm.

L = Length of the sample = 8.5 cm.

A = Cross-sectional area of the sample inside the radial permeameter = 77.37 sq.cm.

$$k_r = \left[\frac{a \log_e \frac{R}{r_o}}{2\pi L}\right] \frac{\log_e \frac{h_o}{h_1}}{t} = \lambda_r \frac{\log_e \frac{h_o}{h_1}}{t}, \quad \lambda_r = \frac{a \log_e \frac{R}{r_o}}{2\pi L} = \frac{1.052 \times 2.04}{2 \times \pi \times 8.5} = 4.02 \times 10^{-2}$$

e	h_o in cm	h_1 in cm	$\frac{h_o}{h_1}$	$\log_e \frac{h_o}{h_1}$	t in sec	$\log_e \frac{h_o}{h_1}$	$K_r \times 10^{-6}$ in cm/sec	$K_{r\,av} \times 10^{-6}$	T°C	$\frac{u_T}{u_{20}}$	$K_r \times 10^{-6}$ 20°C in cm/sec.
0.752	131.3	127.6	1.029	0.0285	903	0.314×10^{-4}	1.262	1.262	17.5	1.06	1.32
	127.3	123.9	1.028	0.0274	900	0.305×10^{-4}					
0.69	131.3	128.1	1.025	0.0246	920	0.267×10^{-4}	1.073	1.055	21	0.985	1.04
	128.0	125.0	1.024	0.0236	918	0.258×10^{-4}	1.04				

void ratios are shown in table 2. In both the cases temperature corrections were applied to get the k_r value at 20°c.

TABLE 2

RADIAL PERMEABILITY OF KAOLINITE (By constant head)

$$k_r = \frac{\log_e \frac{R}{r_o}}{2\pi L H} \frac{Q}{t} = B' \frac{Q}{t} \text{ where } B' = \frac{\log_e \frac{R}{r_o}}{2\pi L H} \text{ \& } H = \text{Constant head applied.}$$

e	Q in c.c.	t in sec.	Q/t	H in cm	B'	$k_r \times 10^{-6}$ in cm/sec	$T^{\circ}C$	$\frac{\mu t}{\mu 20}$	$k_r \times 10^{-6}$ $20^{\circ}C$ cm/sec
0.752	4	900	0.445×10^{-2}	126.6	3.01×10^{-4}	1.34	20	1	1.34
0.69	2	580	0.345×10^{-2}	126.6	3.01×10^{-4}	1.04	19.5	1.01	1.05

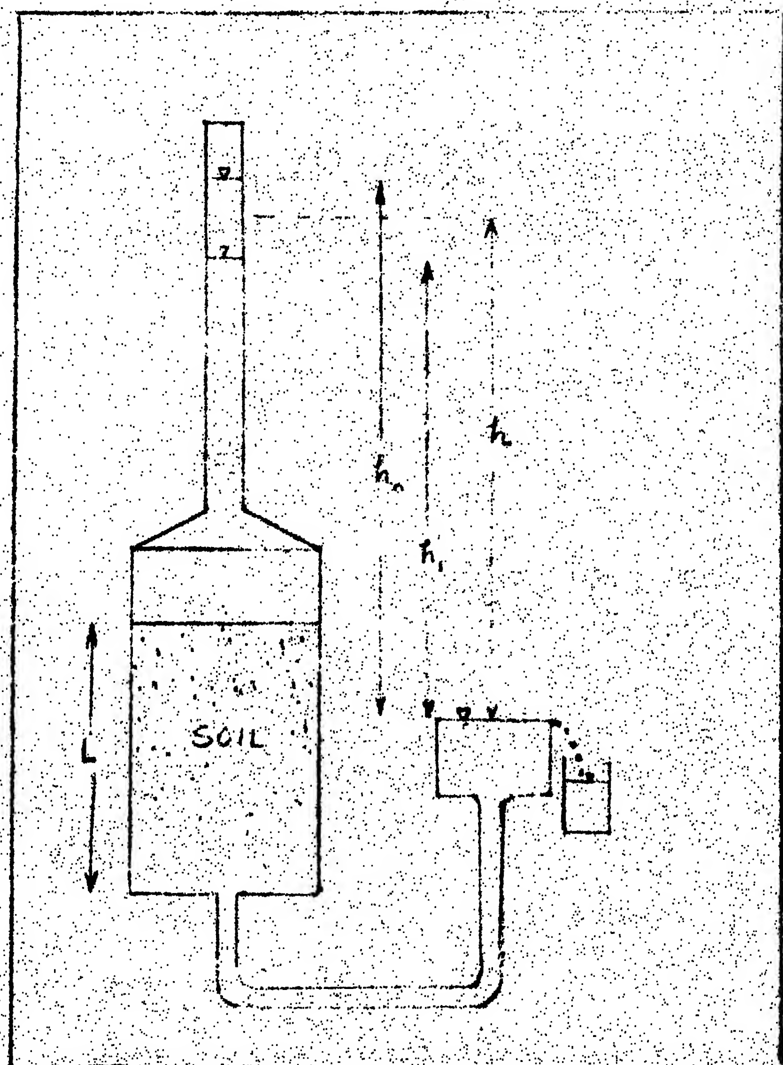
As seen from the tables the agreement between the values of k_r by constant head and variable head is excellent.

3.6 DETERMINATION OF VERTICAL PERMEABILITY:

A usual falling head permeameter arrangement was used for the determination of vertical permeability. It is illustrated diagrammetically in fig 5.

With the area and length of sample and the area of the stand pipe known, and with the time lapse of the initial and final head readings recorded, it is possible to compute the measured permeability, based and derived from Darcy's law from the formula

FIG. 5



AN OUTLINE FOR VERTICAL PERMEAMETER

$$k_v = \frac{aL}{At} \log_e \frac{ho}{h_1} = \lambda_v \frac{\log_e \frac{ho}{h_1}}{t} \quad \ldots\ldots\ldots \quad 3.3$$

where $\lambda_v = \frac{a}{A}L$

k_v = Coefficient of vertical permeability

a = Area of stand pipe

L = Length of sample

t = time lapse

h_o = Initial head

h_1 = Final head.

A photographic view of the radial permeameter designed and developed is shown in photo plate 1 and 2.

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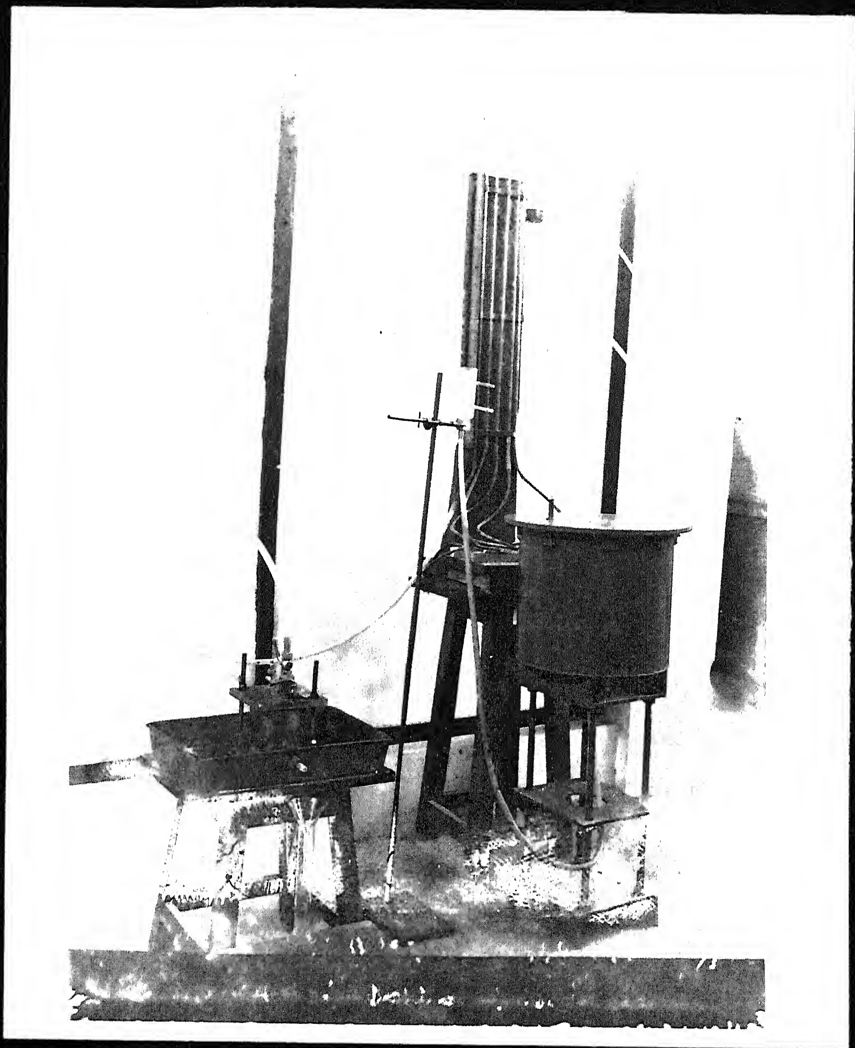
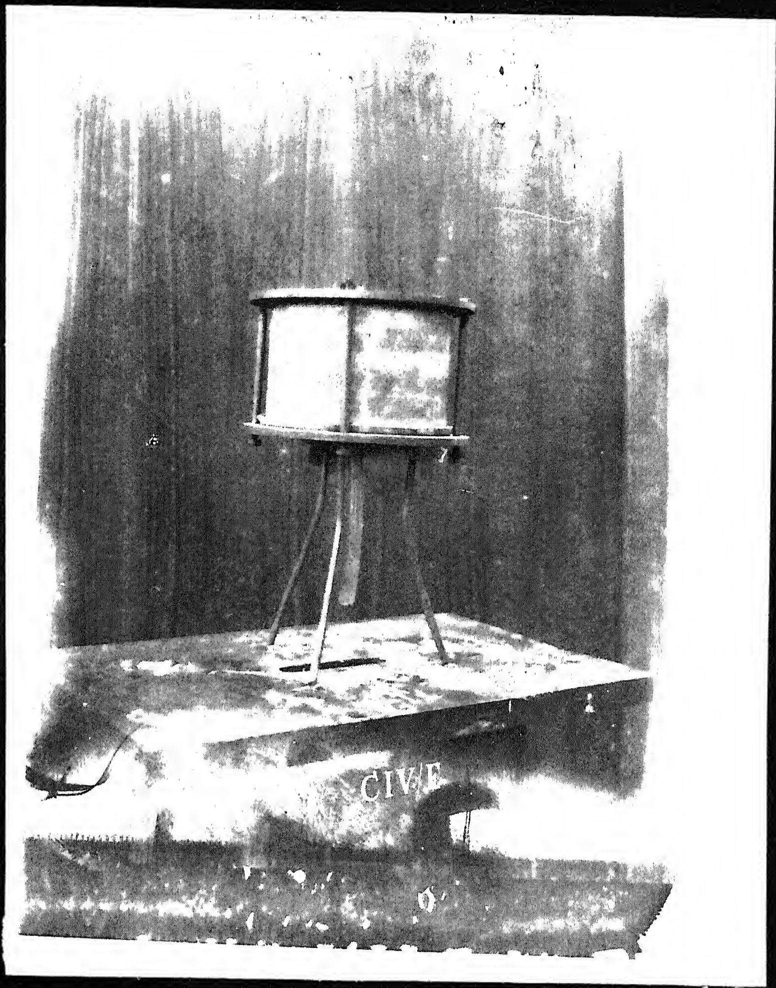
A PHOTOGRAPHIC VIEW OF EXPERIMENTAL SET UP



CYLINDRICAL POROUS SOIL CONTAINER



CHAPTER 4

TESTING PROGRAMMES AND PROCEDURES

4.1 OBJECTIVE AND PROGRAMME:

The main objective of the experimental study was to investigate, within a single layer of Kaolinite

(a) Effect of void ratio, e, on vertical permeability, k_v.

(b) Effect of void ratio, e, on radial permeability, k_r

(c) Effect of void ratio, e, on permeability ratio, r_k.

(d) Effect of structure of a soil on its permeability properties.

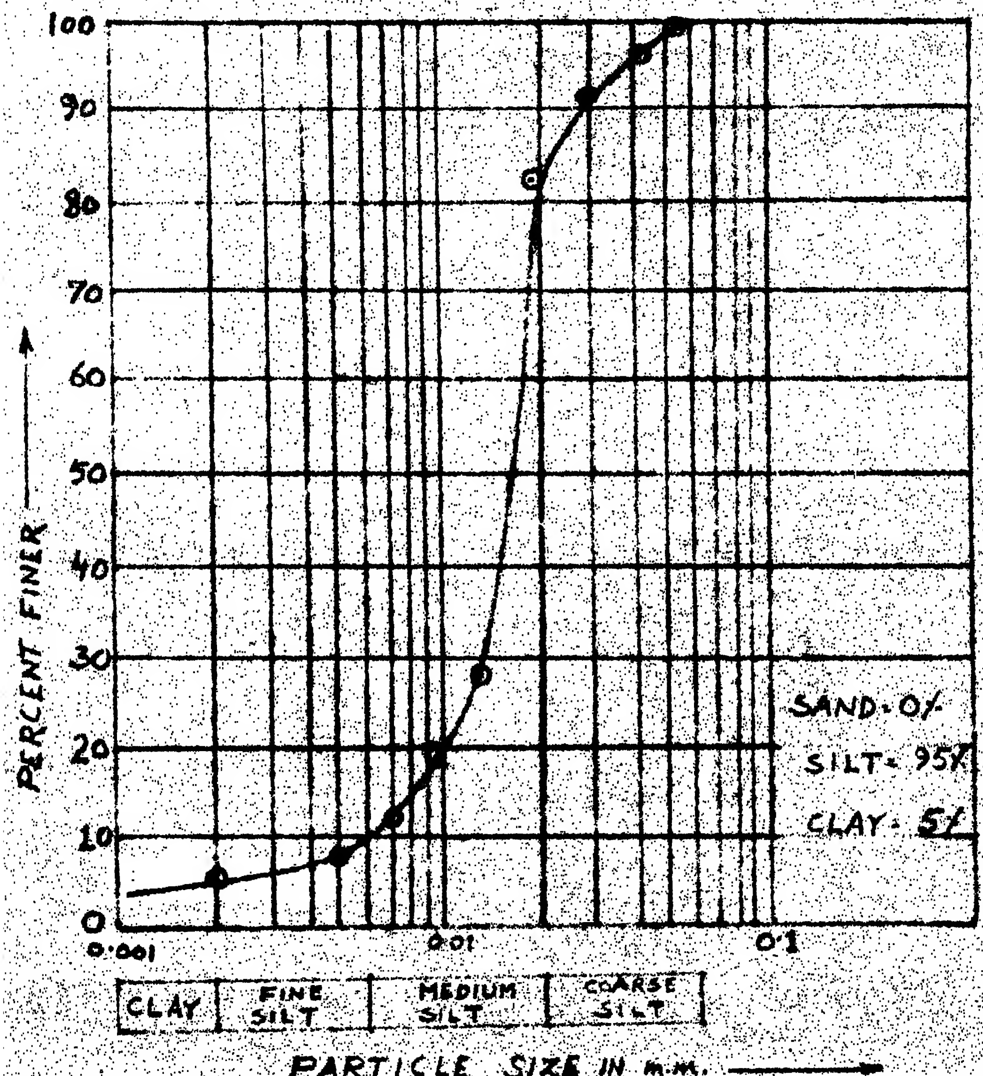
(e) If possible, to establish a suitable structural scale for soil.

In an attempt to answer all these questions, three series of tests were conducted. Each series contained eight tests, four for vertical permeability at different void ratios and four for radial permeability at same or comparable void ratios. The first series was of normal kaolinite, the second series was a comparatively flocculated and the third series was relatively dispersed. How these relative structures were obtained will be described later. Another series of permeability tests on granular soils were conducted with a view to study the effect of shape on permeability of sands and which is separately dealt in chapter 7.

4.2 DESCRIPTION OF SAMPLE TESTED:

The samples tested were of commercially available kaolinite. Whose grain size destribution is shown in fig. 6. The grain size curve shows that the sample is fairly uniform. The specific gravity is found to be 2.5 and liquid limit as 54% with plasticity index as 27%.

FIG. 6



GRAIN SIZE CURVE FOR KAOLINITE

4.3 TECHNIQUES ADOPTED TO GET DIFFERENT VOID RATIOS:

At the begining it was thought that each two permeability tests (one for k_v and other for k_r) will be run at a predetermined void ratio, but from the experience of a few initial tests, it was found very difficult to reach same void ratio for two samples. Hence it was decided that the void ratio will not be fixed earlier and the tests will be conducted at any void ratio within the possible void ratio range and then plotted in the graph to obtain values of k_v and k_r at desired void ratios.

For vertical permeability tests, the void ratio is calculated by measuring the length of the sample in the permeameter. The calculation is simple and runs as follows

$$1+e = 1+\frac{V_v}{V_s} = \frac{V}{V_s} = \frac{V}{\frac{W_s}{G\gamma_w}} = \frac{V\,G}{W_s} \quad (\text{ in C.G. S unit})$$

$$e = \frac{V\,G}{W_s} - 1 = \frac{ALG}{W_s} - 1\,, \quad \text{A for vertical permeameter is 80 sq.cm.}$$

$$e = \frac{80\,G}{W_s}L - 1 \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad 4.1$$

Now, in this expression, G for a particular soil is known W_s, the quantity of solid taken is also known and hence by knowing L, we can find out the void ratio e.

A definite quantity of dry soil W_s(with a known initial moulding water content) is taken inside the permeameter and the sample is then compacted by a static compacter (procedure is described in detail later). The length of the sample is then accurately measured and the corresponding void ratio is found, In measuring the length, for each sample and each void, three scale measurements are taken and then the average of it is taken as accepted value.

As a check, the length of measurements were taken one at the begining and one at the end of test to detect any possible change of void ratio during test due to swelling. But for kaolinite no swelling was observed

For radial permeability, void ratios are calculated using the same expression (4.1) execpt that 'A' is 77.37 sq.cm. For the first test, the whole porous cylinder is completely filled with soil. To get another void ratio, the sample is compressed and the new length of the sample and the new value of 'e' found from the expression (4.1).

4.4 COMPACTION PROCEDURE FOLLOWED:

At the begining, proctor compaction was thought of, in the permeameter itself and tried for few experiments, but it was found difficult and trouble some. So the idea was dropped and static compaction was resorted to.

For this purpose, an wooden block of 10 cm. diam. 5 cm thick with a central hole of 1.4 cm diameter was prepared. For compacting the sample, in the radial permeameter unit , this block was kept over the soil and an compressive load was applied to reduce the length of the sample and hence to reduce the void ratio.

For compacting the sample in the vertical permeameter, an exactly same type of wooden block without the annular hole was used.

4.5 PREPARATION AND SATURATION OF THE SAMPLE:

Required amount of airdried kaolinite was taken and with it 50% water (by Vol.) was added and mixed thoroughly to prepare a good soil water mix or slurry. This slurry was poured inside the permeameter units and compacted to some length. For the second series of tests, 5% (by wt) pure Nacl was added with the slurry and kept 48 hours

for reaction to get a comparatively flocculated structure. For the 3rd series of tests, similary 5% (by wt.) Na oxalate was mixed thoroughly and kept for the same 48 hrs. to get a comparatively dispersed structure of the sample.

To saturate, the samples were subjected to 25 ft head of water for complete 24 hrs. Throughout all the tests, tap water was used as a permeant to take the advantage of 25' head of water of the overhead tank which was very essential to saturate the samples. An analysis of tap water is given in table 3 below*.

TABLE 3

TAP WATER ANALYSIS

Physical properties		Chemical properties	
Turbidity	= 0 mg/l	pH value	= 8.35
Colour	= no colour	Total alkalinity	= 540 mg/l.
		Total Hardness	= 200 mg/l.
Taste	= no taste	C.O.D.	= 0 mg/l
odour	= no odour	B.O.D.	= 0 mg/l
		Ammonia Nitrogen	= 0 mg/l
		Organic Nitrogen	= 0 mg/l
		Nitrate Nitrogen	= 0 mg/l
		Iron	= 0.68 mg/l
		Sulphates	= 50 mg/l
		Chlorides	= 80 mg/l
		Residual cl_2	= 0.07 mg/l

*Thanks to sanitary engg. Deptt. for supplying the data.

4.6 TEMPERATURE CORRECTION:

Temperature was found to be fluctuating throughout the day. To minimise the effect of temperature on the test results, all the experiments were conducted in between 7 A.M. to 10 A.M. and again from 7.30 P.M. to 10.30 P.M. during which temperature fluctuations was observed to be minimum ($\pm$ 5^{o}c). Ultimately all the permeability values were subjected to temperature correction to get the permeability values at 20^{o}c. The temperature correction applied was (4)

$$K_{20} = \frac{\mu_t}{\mu_{20}} \frac{\gamma_{20}}{\gamma_t} K_T$$

$$\approx \frac{\mu_t}{\mu_{20}} K_T, \text{ as } \frac{\gamma_{20}}{\gamma_T} \text{ does not affect the result}$$

within slide rule accuracy.

. . . . 4.2

Frequent temperature measurements were taken during the tests.

4.7 GENERAL PRECAUTIONS TAKEN:

Following, are some of the points which were strictly followed during testing.

a. Before taking any reading, in all the tests, sufficient time was allowed to reach the steady state for the soil water system.

b. For any particular void ratio, 2 permeability readings were taken and average 2 reading (not differing by more than 5%) was taken as the final result.

c. Before starting any test all the connecting tube lines were deaired by sending jets of water through it, under high pressure.

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CHAPTER 5

RESULTS & INTERPRETATIONS

5.1 TESTS WITH KAOLINITE:

As explained earlier a total of eight tests were conducted four for k_r and four for k_v at different void ratios. The results of k_r tests are shown in table 4 and that of k_v in table 5.

The results of table 4 and 5 are plotted and is shown in figure 7.

As seen from fig.7, the value of k_r is always more than k_v at all void ratios which was expected because of flaky shape of all particles in the kaolinite sample, and its orientation with respect to vertical and horizontal axis.(8). If a material is composed of flake shaped materials, than under the pressure the individual particles will have a tendency to orient themselves such that their greater lengths will be perpendicular to load. The process can be explained with the help of fig. 8.

This tendency of individual particles to orient themselves in parallel direction and perpendicular to the load can also be explained from the stress point of view. In nature every matter tries to keep itself in the minimum state of energy or stress level and hence in this case each individual particle orients in such a way that it gives it's maximum area to the direction of loading.

To explain the phenomena of particle orientation let's introduce a term called "Degree of parallelism (η)" which will indicate the stage of parallelism of the particles with respect

TABLE 4

RADIAL PERMEABILITY OF KAOLINITE

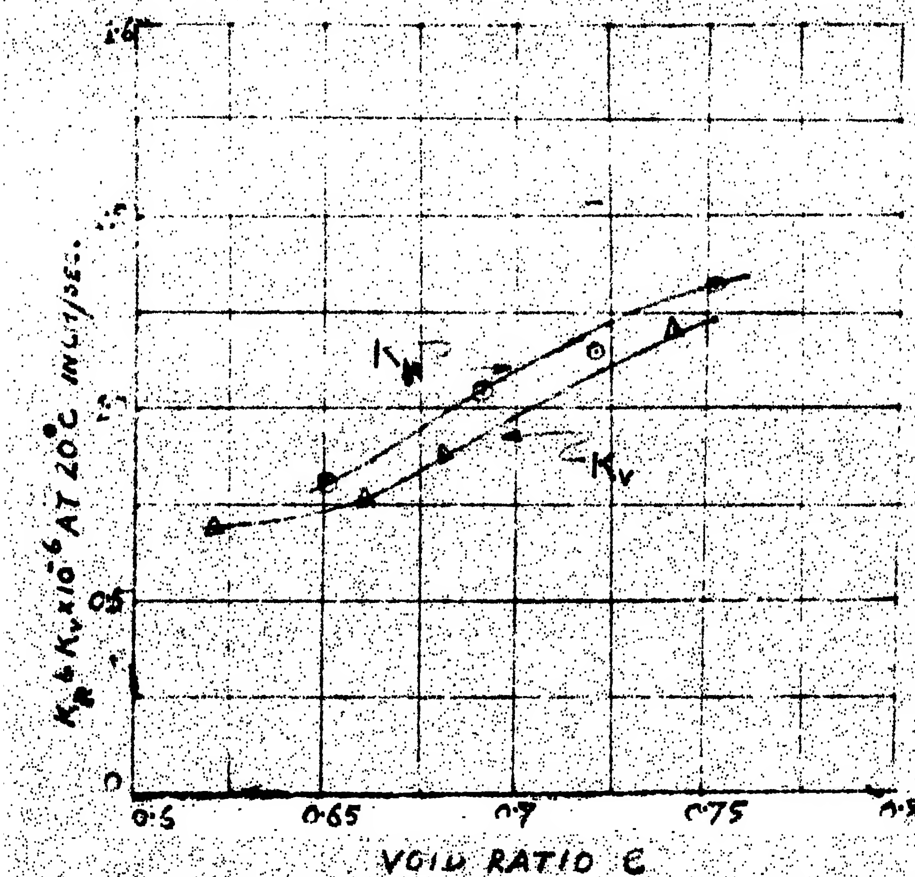
e	h_o in cm.	h_1 in cm	h_o/h_1	$\log_e h_o/h_1$	t in secn	$\frac{\log_e \frac{h_o}{h_1}}{t}$	λ_r	$k_r \times 10^{-6}$ in sm/sec.	$k_{rav} \times 10^{-6}$ in cm/sec	T in ^oc	$\frac{\mu_t}{\mu_{20}}$	$k_r \times 10^{-6}$ at $20^o c$ in cm/sec.
0.752	131.3	127.6	1.029	0.0284	903	0.314×10^{-4}	4.025×10^{-2}	1.262	1.245	17.5	1.06	1.32
	127.3	123.9	1.028	0.0274	900	0.305×10^{-4}		1.23				
0.72	124	121	1.024	0.0236	900	0.262×10^{-4}	-do-	1.002	1.002	15	1.134	1.14
	121.1	118.1	1.024	0.0236	900	0.262×10^{-4}		1.002				
0.69	131.3	128.1	1.025	0.0246	920	0.267×10^{-4}	-do-	1.073	1.053	21	0.985	1.04
	123.0	125	1.024	0.0236	918	0.258×10^{-4}		1.0[illegible]				
0.65	136.3	134.2	1.017	0.0166	922	0.18×10^{-3}	-do-	0.725	0.73	16	1.105	0.806
	132	130	1.017	0.0166	903	0.183×10^{-3}		0.735				

TABLE 5

VERTICAL PERMEABILITY OF KAOLINITE

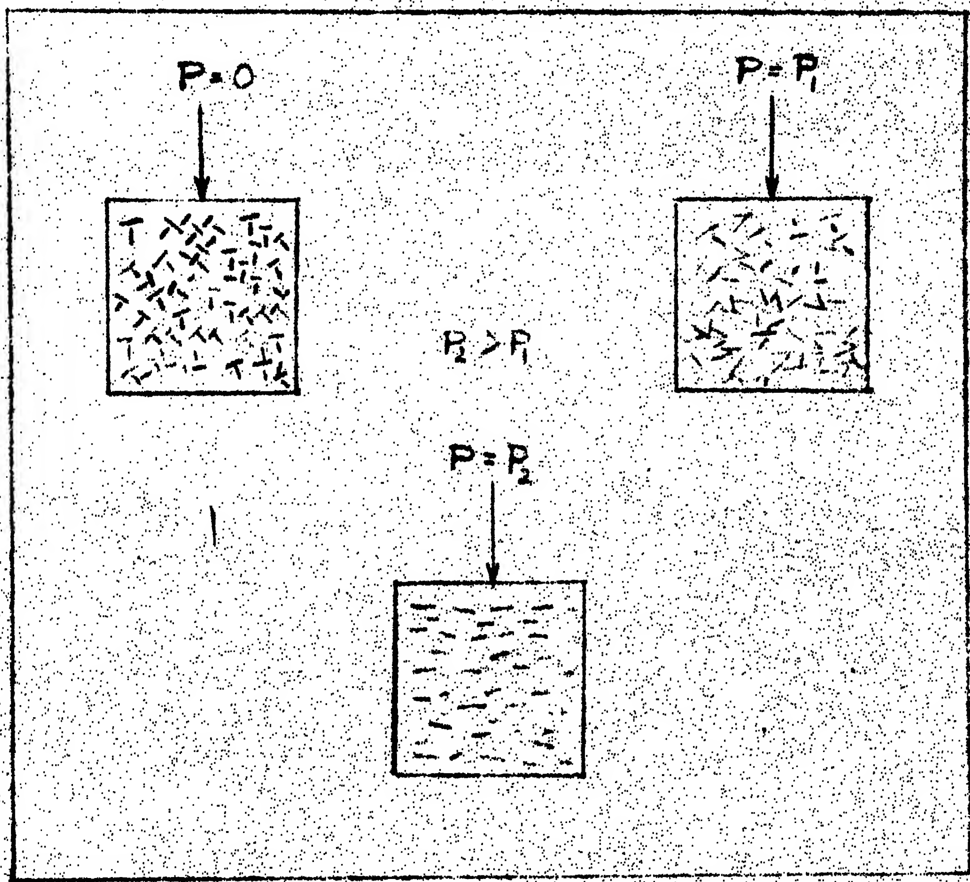
L in cm	e	h_o in cm	h_1 in cm	h_o/h_1	$\log_e \frac{h_o}{h_1}$	t in sec.	$\frac{\log_e \frac{h_o}{h_1}}{t}$	λ_v	$k_v \times 10^{-6}$ in cm/sec.	$k_{vav} \times 10^{-6}$ in cm/sec.	T in ^oc	$\frac{\mu t}{\mu_{20}}$	$k_r \times 10^{-6}$ at 20^oc in cm/sec.
8.7	0.74	194.6	192.7	1.01	0.0099	900	0.11×10^{-4}	11.47×10^{-2}	1.26	1.26	21	0.975	1.23
		192.6	190.6	1.01	0.0099	900	0.11×10^{-4}	11.47×10^{-2}	1.26				
8.4	0.68	195.4	194.0	1.007	0.0067	902	0.0742×10^{-4}	11.2×10^{-2}	0.831	0.831	17	1.078	0.888
		193.9	192.5	1.007	0.0067	904	0.0742×10^{-4}	11.2×10^{-2}	0.831				
8.3	0.66	190.2	188.8	1.007	0.0067	900	0.0744×10^{-4}	10.92×10^{-2}	0.82	0.818	23.5	0.935	0.765
		188.7	187.45	1.007	0.0067	905	0.074×10^{-4}	10.92×10^{-2}	0.817				
8.1	0.62	194	191.9	1.011	0.0109	1802	0.0604×10^{-4}	10.65×10^{-2}	0.643	0.643	15.5	1.12	0.72
		191.9	189.8	1.011	0.0109	1800	0.0604×10^{-4}	10.65×10^{-2}	0.643				

FIG 7



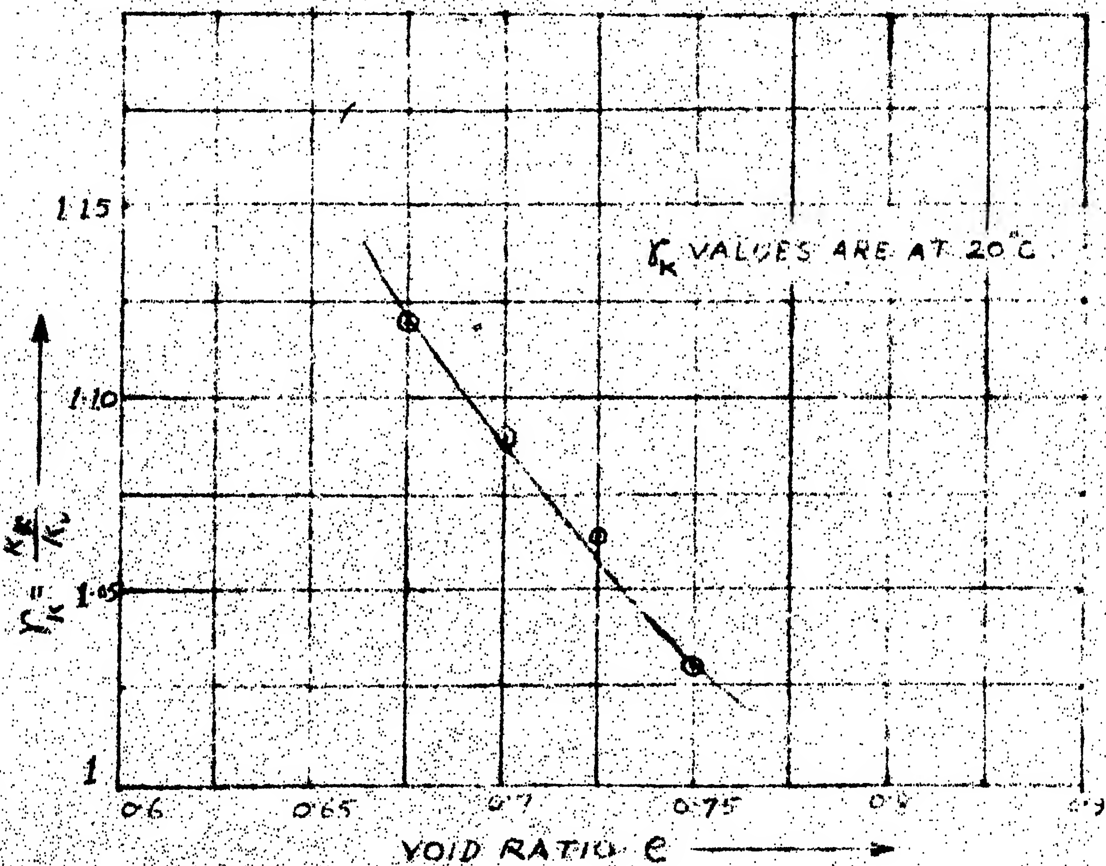
VARIATION OF K_R & K_V OF KAOLINITE WITH VOID R

FIG. 8



A SCHEMATIC REPRESENTATION OF PARTICLE ORIENTATION UNDER LOADING.

FIG. 9



VARIATION OF PERMEABILITY RATIO OF KAOLINITE WITH VOID RATIO.

to the horizontal and vertical direction. As seen from the fig. 8 degree of parallelism 'η' increases with increasing overburden load or decreasing void ratio. Again as η increases the tortuosity of the flow path in the horizontal direction will decrease with respect to the vertical direction and thereby increasing k_r more than k_v.

From the fig.7 k_r and k_v values are measured for the void ratio of 0.675, 0.7, 0.725 and 0.75 and the permeability ratio k_r/k_v ($= \gamma_k$) are plotted against the void ratio as shown in fig. 9. As seen from fig. 9 γ_k decreases with increasing void ratio and logically it seems that at large void ratios the value of γ_k will tend to 1 and for a small void ratio γ_k will be large. The trend of e vs. γ_k curve is fully consistent with the concept of degree of parallelism because as η increases i.e. as void ratio decreases the horizontal flow path will be less and less giving rise to greater γ_k. At very low void ratio, the permeability ratio γ_k may attain a limiting value $\gamma_{k\ max}$.

5.2 TESTS WITH FLOCCULATED AND DISPERSED KAOLINITE:

Samples were prepared with 5% (by wt.) Nacl. as flocculating agent and also with 5% (by wt) sodium oxalate as dispersing agent to study the effect of flocculation and dispersion on radial and vertical permeability. The details of sample preparation has already been given in section 4.5. The test results for kaolinite with 5% sodium oxalate as dispersing agent are shown in table 6 and 7 and that of 5% Na cl as flocculating agent are shown in Table 8 and 9.

TABLE 6

RADIAL PERMEABILITY OF KAOLINITE WITH 5% Na OXALATE

λ_r	e	h_o in cm	h_1 in cm	h_o/h_1	$\log_e \frac{h_o}{h_1}$	t in sec	$\frac{\log_e \frac{h_o}{H_1}}{t}$	K_r x10^{-6} in cm/sec.	$k_{r\,av}$ x10^{-6} in cm/sec.	T in ^{o}c	$\frac{u_t}{u_{20}}$	k_r x10^{-6} at 20^{o}c in cm./sec.
	0.842	138.8	131.8	1.053	0.0517	900	0.574x10^{-4}	2.31	2.31	15	1.134	2.62
		131.6	125	1.053	0.0517	900	0.574x10^{-4}	2.31				
	0.76	135	130	1.038	0.0372	900	0.413x10^{-4}	1.665	1.665	15	1.134	1.89
4.025 x10^{-2}		130	125.1	1.038	0.0372	900	0.413x10^{-4}	1.665				
	0.68	138.8	135.7	1.023	0.0227	900	0.252x10^{-4}	1.015	1.015	14	1.165	1.182
		135.5	132.5	1.023	0.0227	900	0.252x10^{-4}	1.015				
	0.628	136	133.9	1.016	0.0156	900	0.1732x10^{-4}	0.695	0.695	16	1.105	0.77
		133.9	131.8	1.016	0.0156	900	0.1732x10^{-4}	0.695				

TABLE 7

VERTICAL PERMEABILITY OF KAOLINITE WITH 5% Na OXALATE

L in cm	e	h_o in cm	h_1 in cm	h_o/h_1	$\log_e \frac{h_o}{h_1}$	t in sec	$\frac{\log_e \frac{h_o}{h_1}}{t}$	λ_v	$k_v \times 10^{-6}$ in cm/sec	$k_{vav} \times 10^{-6}$ in cm./sec	T in °c	$\frac{\mu_t}{\mu_{20}}$	$k_v \times 10^{-6}$ at 20°c in cm/sec.
9.8	0.867	190.2	186.4	1.021	0.0208	900	0.231×10^{-4}	12.9×10^{-2}	3.08	3.08	19.5	1.01	3.10
		186.3	182.6	1.021	0.0208	900	0.231×10^{-4}		3.08				
9.4	0.79	193.9	191.1	1.015	0.0147	900	0.1632×10^{-4}	12.39×10^{-2}	2.02	2.02	20.5	0.96	1.94
		191.0	188.1	1.015	0.0147	900	0.1632×10^{-4}		2.02				
8.05	0.725	193.0	191.1	1.01	0.0099	1200	0.0825×10^{-4}	11.9×10^{-2}	0.982	0.982	17	1.078	1.058
		189.7	187.8	1.01	0.0099	1200	0.0825×10^{-4}		0.982				
7.95	0.7	193.8	192.6	1.007	0.0067	900	0.0745×10^{-4}	11.79×10^{-2}	0.876	0.876	17.5	1.06	0.93
		192.6	191.3	1.007	0.0067	900	0.0745×10^{-4}		0.876				

TABLE 8

RADIAL PERMEABILITY OF KAOLINITE WITH 5% Naol

λ_r	e	h_o in cm.	h_1 in cm.	h_o/h_1	$\log_e \frac{h_o}{h_1}$	t in sec.	$\frac{\log_e \frac{h_o}{h_1}}{t}$	$k_r \times 10^{-6}$ in cm/sec.	$k_{r\,av} \times 10^{-6}$ in cm/sec	T in °c	$\frac{\mu_t}{\mu_{20}}$	$k_r \times 10^{-6}$ at 20°c in cm/se
4.025×10^{-2}	0.76	134.1	131.3	1.021	0.0208	900	0.231×10^{-4}	0.93	0.93	20	1	0.93
		131.2	128.6	1.021	0.0208	900	0.231×10^{-4}	0.93				
	0.715	138.4	134.9	1.026	0.0255	1500	0.17×10^{-4}	0.685	0.672	18	1.05	0.719
		134.9	131.6	1.025	0.0246	1500	0.164×10^{-4}	0.66				
	0.69	135.7	132.4	1.024	0.0236	1500	0.157×10^{-4}	0.632	0.62	18	1.05	0.642
		132.4	129.4	1.023	0.0227	1500	0.1515×10^{-4}	0.61				

TABLE 9

VERTICAL PERMEABILITY OF KAOLINITE WITH 5% Nacl.

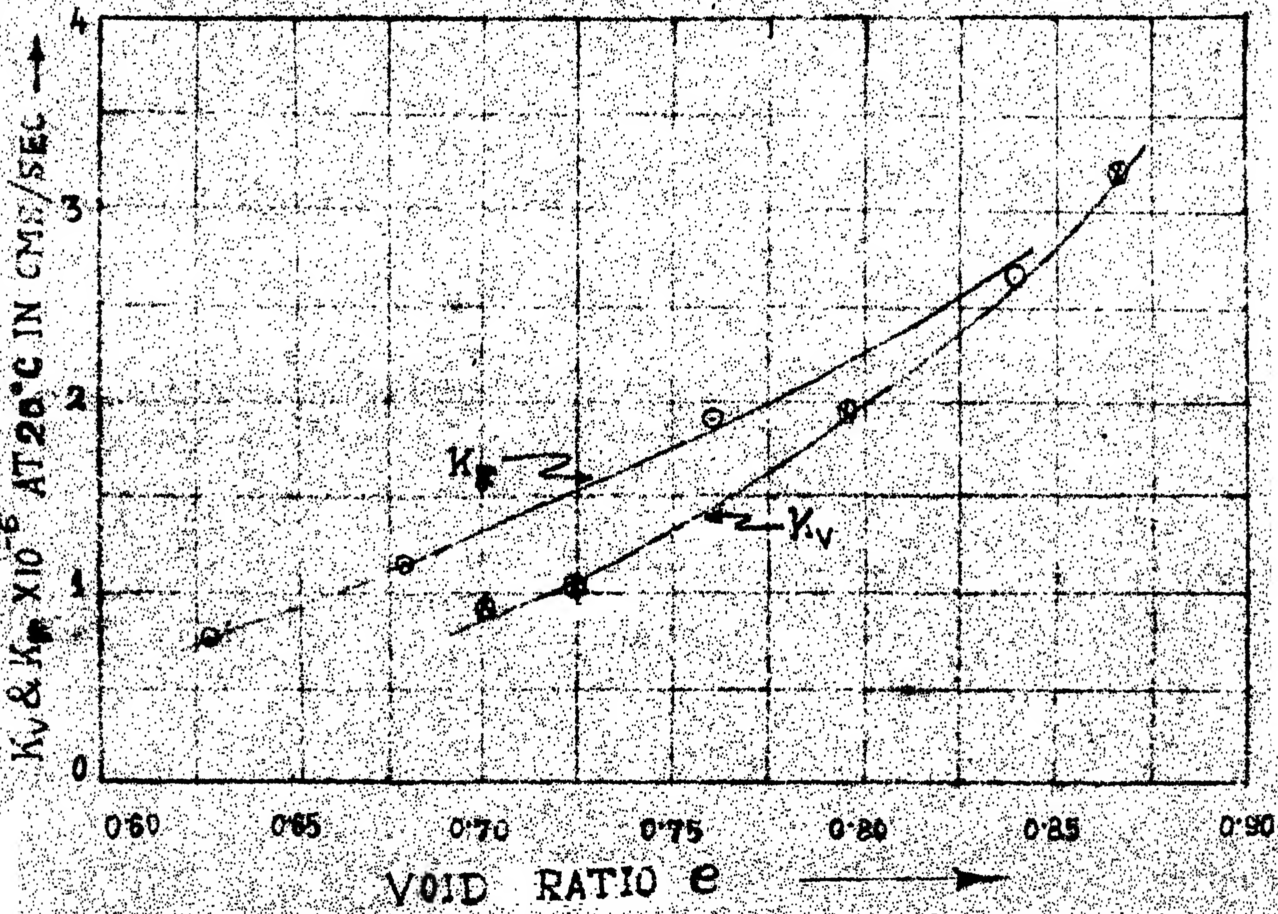
L in. cm	e	h_o in cm	h_1 in cm	h_o/h_1	$\log_e \frac{h_o}{h_1}$	t in sec	$\frac{\log_e \frac{h_o}{h_1}}{t}$	λ_v	$k_v \times 10^{-6}$ in cm/sec	$k_{vay} \times 10^{-6}$ in cm/sec.	T in ^oc	$\frac{n_t}{n_{20}}$	$k_v \times 10^{-6}$ at 20^oc in cm/sec.
9.2	0.84	192.0	189.3	1.014	0.0137	900	0.1521×10^{-4}	12.1×10^{-2}	1.64	1.84	23.5	0.915	1.685
		189.3	186.8	1.014	0.0137	900	0.1521×10^{-4}		1.84				
8.8	0.76	196.0	194.7	1.007	0.0067	900	0.0745×10^{-4}	11.59×10^{-2}	0.864	0.864	17.5	1.06	0.915
		194.7	193.5	1.007	0.0067	900	0.0745×10^{-4}		0.864				
8.5	0.7	138.7	137.3	1.009	0.0086	1500	0.0574×10^{-4}	11.2×10^{-2}	0.642	0.642	17.5	1.06	0.681
		137.3	136.0	1.009	0.0086	1500	0.0574×10^{-4}		0.642				
8.35	0.67	136.0	134.6	1.01	0.0099	1800	0.055×10^{-4}	11×10^{-2}	0.605	0.605	22	0.952	0.576
		134.0	132.7	1.01	0.0099	1800	0.055×10^{-4}		0.605				

Results of permeability tests with 5% Na Oxalate are plotted and shown in figure 10. From this plotted curves, values of k_r and k_v at void ratios of 0.7, 0.725, 0.75, 0.775, 0.80, 0.825 & 0.85 are measured and as previously a plot void ratio vs. γ_k is obtained and is is shown in fig. 11. Similarly, curves were also prepared for kaolinite with 5% Nacl and are shown in fig. 12 and 13. As seen from the fig. 12 and 13, the difference between k_r and k_v at the void ratio range tested is almost nil. The reason is, perhaps, degree of parallelism 'η' changes very slightly with the void ratio (within the void ratio range tested) because of Nacl, which is used as flocculating agent. But kaolinite with 5% dispersing agent shows just the opposite result as seen from fig. 10 and 11 which was expected because of dispersing effect of Na oxalate which separates each particle from one another and thereby cause an appreciable change in degree of parallelism η with void ratio e.

To have a clear picture of relative effect of flocculating and dispersersing agent on k_r, k_v and γ_k of kaolinite at different void ratios, fig. 7 to 13 are used to get k_r, k_v and γ_k values at selective void ratios and are shown in fig. 14, 15 and 16.

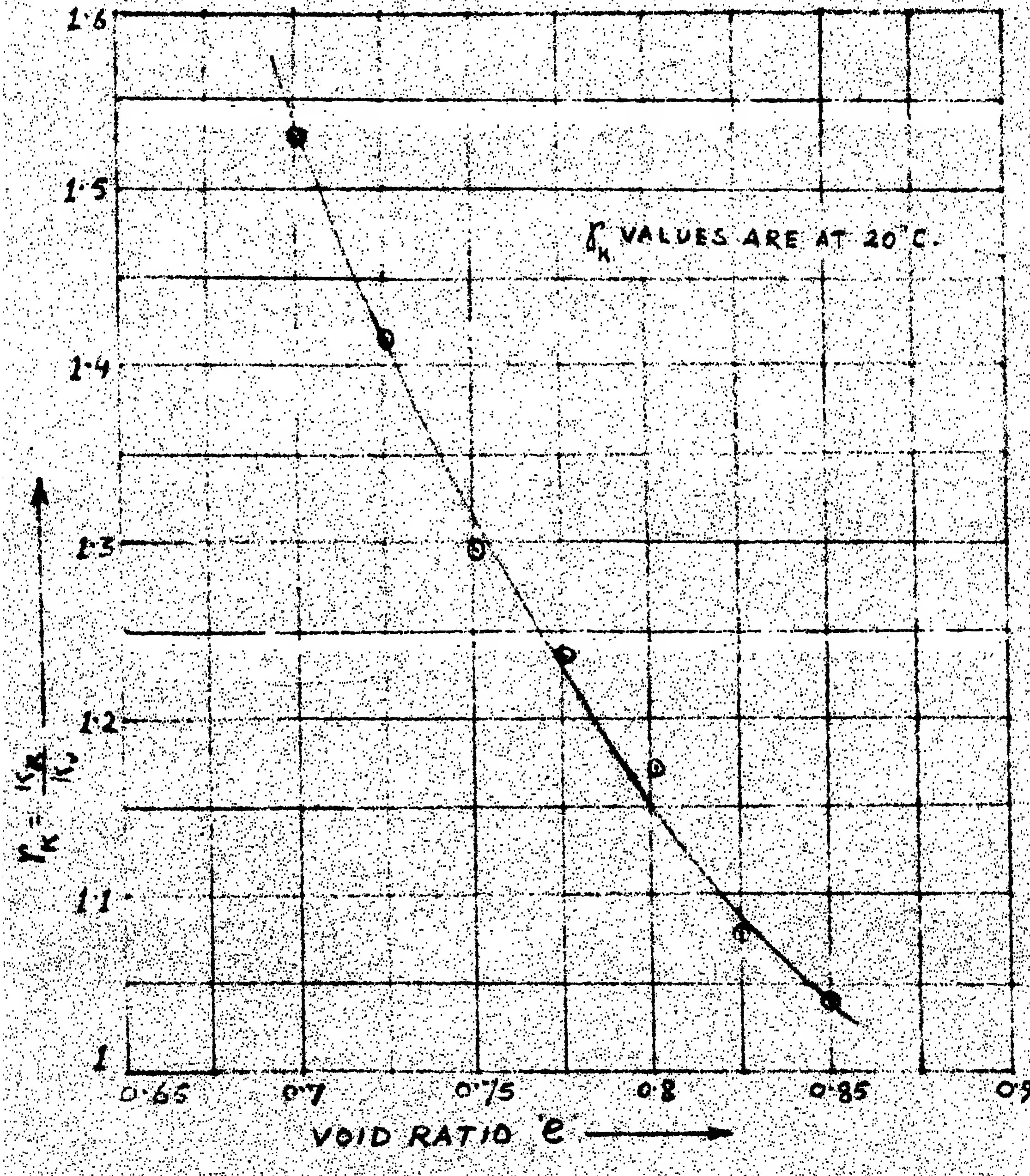
As seen from fig. 14 and 15, within the void ratio range of 0.65 to 0.80, the variation of k_r and k_v is almost linear and again at all void ratios, kaolinite with 5% dispersing agent have higher k_r and k_v value than pure kaolinite where as kaolinite with 5% aggregator have always lesser k_r and k_v values than pure kaolinite. Fig. 16 clearly shows that at all void ratios permeability ratio γ_k has

FIG. 10



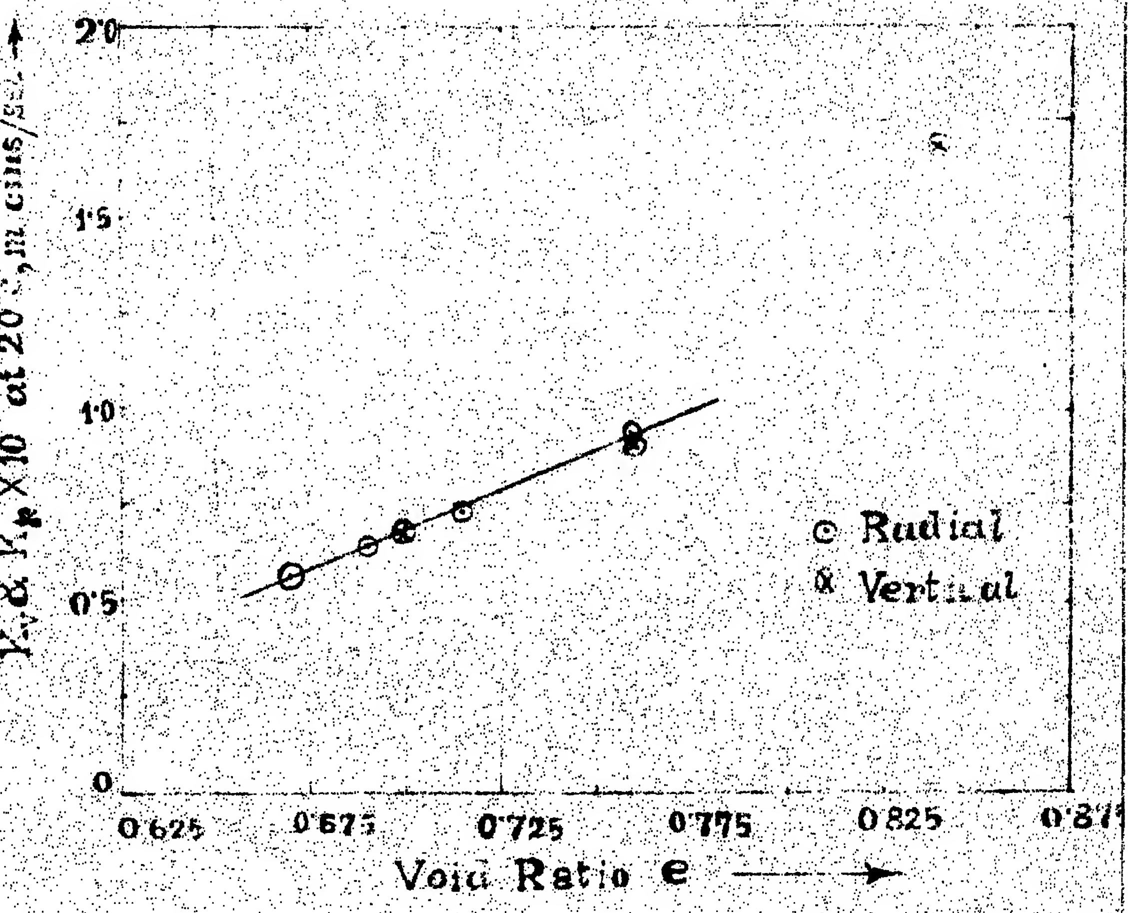
VARIATION OF K_V AND K_R OF KAOLINITE WITH 5% DISPERSING AGENT WITH VOID RATIO.

FIG. 12



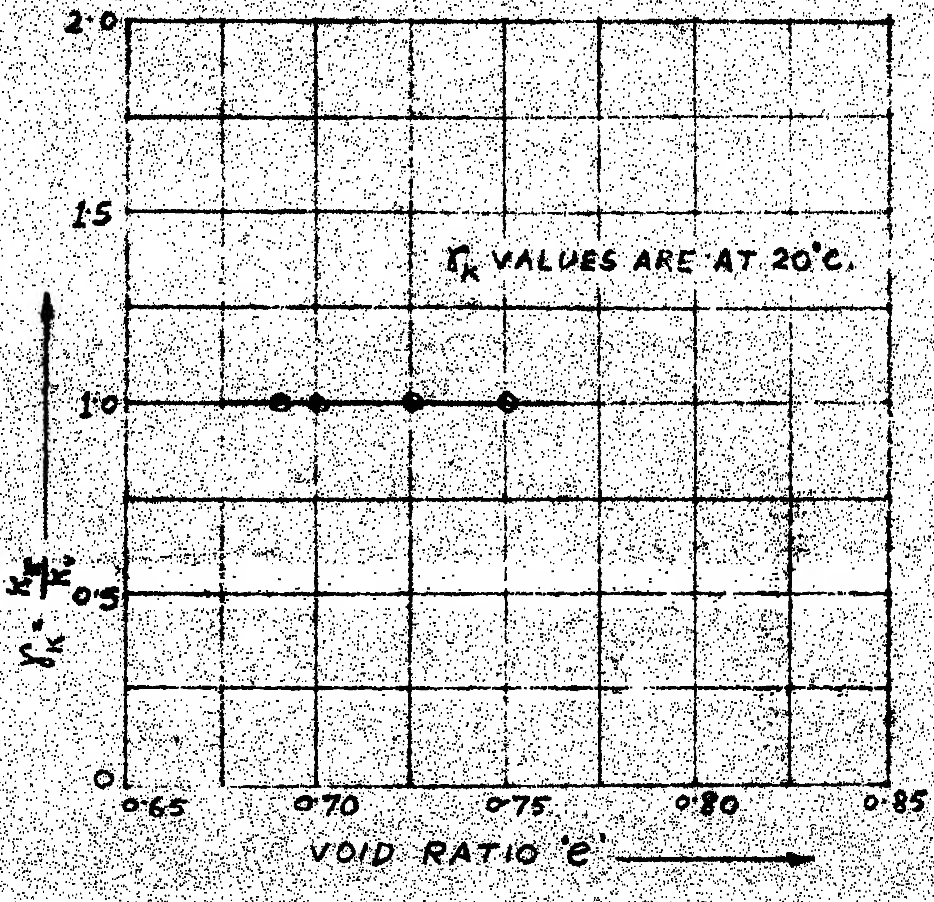
VARIATION OF γ_K OF KAOLINITE WITH 5% DISPERSING AGENT WITH VOID RATIO

Fig. 12



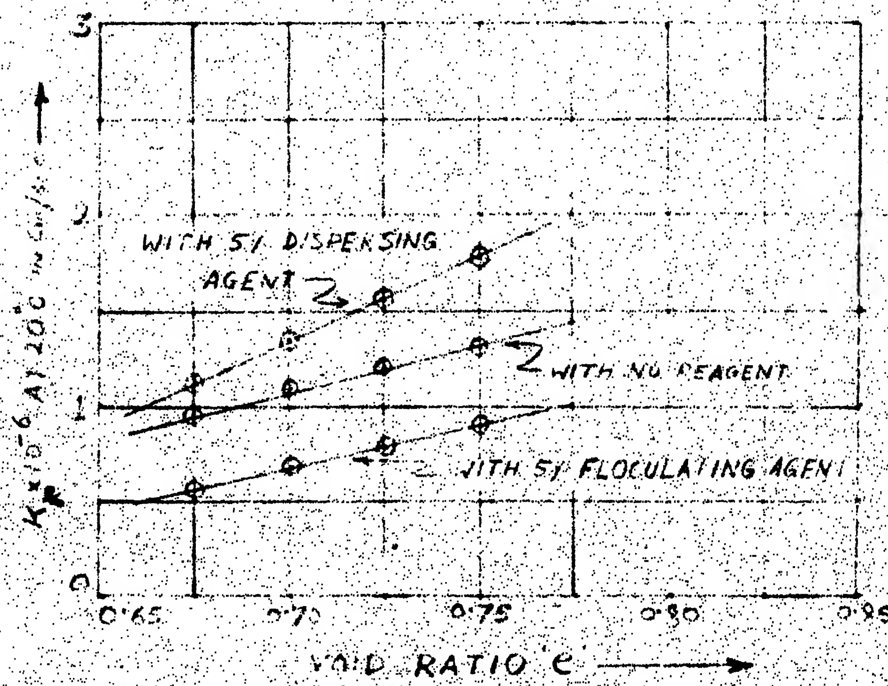
Variation of K_R and K_V of Kaolinite with 5% Flocculating agent with Void Ratio

FIG. 13



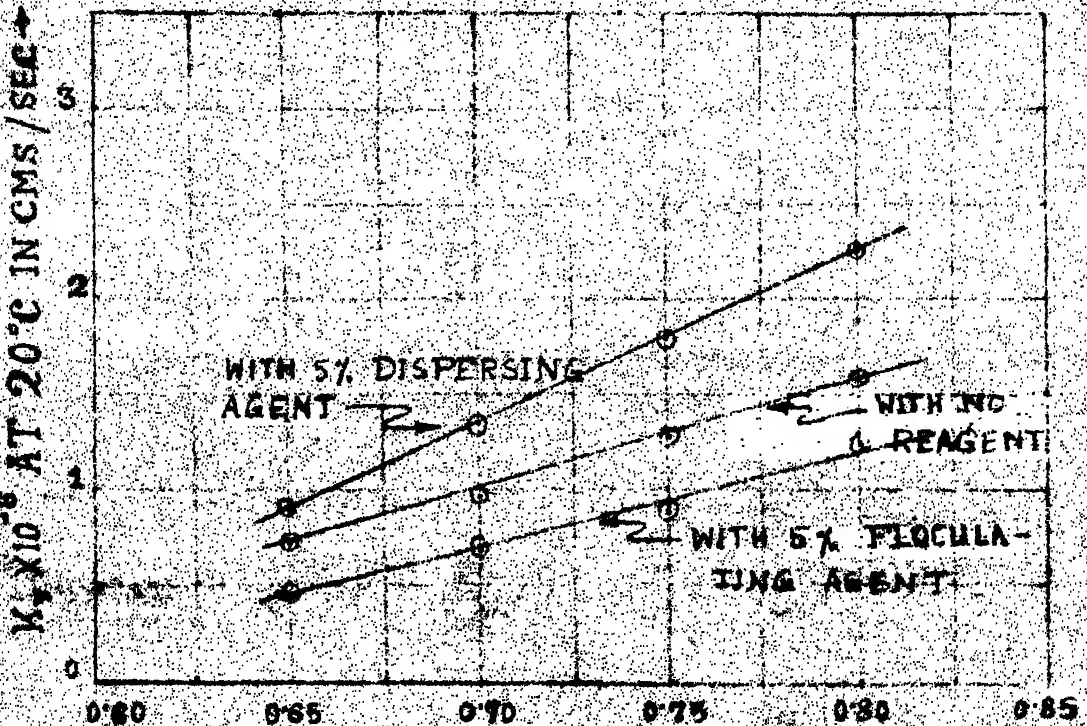
VARIATION OF γ_K OF KAOLINITE WITH 5% FLOCCULATING AGENT WITH VOID RATIO.

FIG. 14



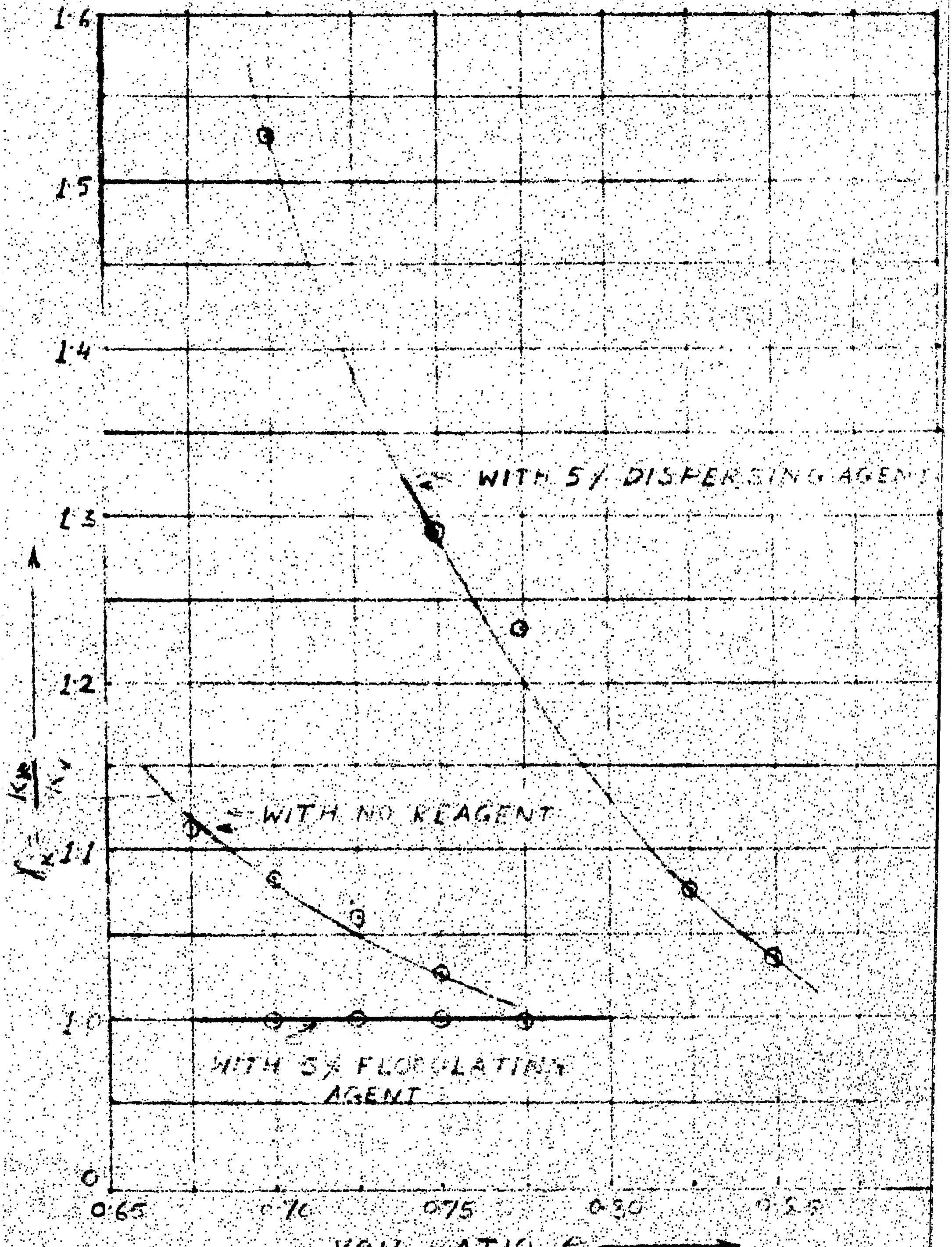
CHANGE IN K_{θ} WITH VOID RATIO FOR DIFFERENT PERCENT DISPERSING AND FLOCULATING AGENT

FIG. 16



CHANGES IN K_V WITH VOID RATIO FOR DIFFERENT PERCENTAGE OF FLOCULATING AND DISPERSING AGENTS

FIG. 16



EFFECT OF PERCENTAGE OF DISPERSING & FLOCULATING AGENT ON γ_K AT DIFFERENT VOID RATIOS

higher value for kaolinite with 5% dispersering agent than pure kaolinite where as kaolinite with 5% flocculating agent has always lesser value than the same pure kaolinite. This phenomena can perhaps be explained also from the concept of degree of parallelism 'η'. At a particular void ratio, degree of parallelism of kaolinite with 5% dispersering agent is perhaps highest and lowest for kaolinite with 5% flocculating agent . And the degree of parallelism of pure kaolinite is samething in between.

The majority of the test results shows that permeability ratio γ_k varies inversely with e

or $\gamma_k \propto \frac{1}{e}$

or $\gamma_k e$ = const. say ψ 5.1

The constant ψ is a function of soil type and its structure or degree of parallelism.

CHAPTER 6

A HYPOTHESIS FOR STRUCTURAL SCALE OF SOILS

6.1 USE AND IMPORTANCE OF STRUCTURAL SCALE:

Although the fact that the structure of soils, plays an important role, on permeability, shear strength, compaction and consolidation was felt long back, there was no suitable method to find out and know, the exact structure of any soil specimen. Uptill now, the structure of any soil was only estimated in the relative sense i.e. maximum we could say was "this sample is more flocculated or dispersed than that sample". Lambe(5) seriously thought of assigning a particular number to a particular soil specimen to denote its position in the structural scale, -but no suitable method and theory was available to fix this structural scale. Here, a very simple hypothesis for structural scale of soils is presented.

6.2 BASIC ASSUMPTION AND DERIVATION OF PROPOSED EQUATION:

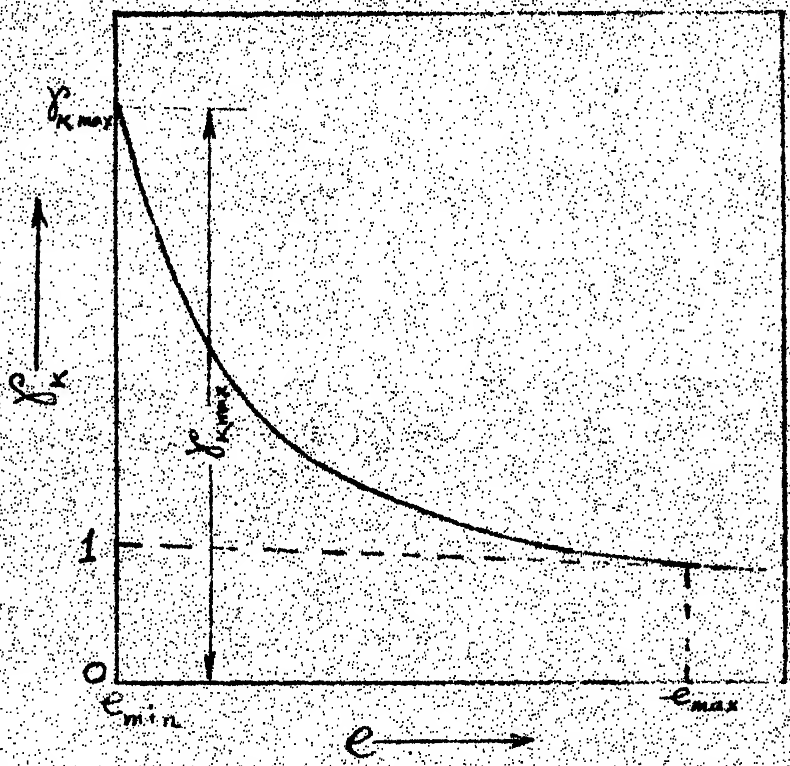
From the concept of degree of parallelism and the experimental results (fig. 16), it is clear that permeability ratio γ_k is inversely proportional to the void ratio of the sample or,

$$\gamma_k \propto \frac{1}{e} \quad \dots\dots\dots\dots\dots\dots \quad 6.1$$

which means and also supported by our experimental results the shape of γ_k vs. e curve will be like as shown in fig. 17.

As originally hinted by Lambe (5), we can say that permeability ratio γ_k is directly proportional to the some power of degree of parallelism or

FIG. 17



A TYPICAL e VS. γ_K CURVE.

$$\gamma_k \propto \eta^{\alpha} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad 6.2$$

where η = void ratio dependent degree of parall-eelism.

α = Some positive number to be determined.

From assumptions (6.1) and (6.2)

$$\gamma_k \propto \frac{\eta^{\alpha}}{e}$$

$$\text{or } \gamma_k = A\,\frac{\eta^{\alpha}}{e} \quad \ldots\ldots\ldots\ldots\ldots \quad 6.3$$

for $e_{min} < e < e_{max}$

where A is a constant of proportionality to be determined.

Minimum void ratio e_{min} is defined as a minimum void ratio possible which can be obtained in the laboratory and where the permeability ratio may obtain a limiting value γ_{kmax} as seen from fig. 17. Similarly e_{max} is the loosest void ratio possible for a soil and where the permeability ratio value will be almost equal to one.

Now, to find out A and α from equation (6.3) we can use the following two boundary conditions

i. at $e = e_{max}$, $\gamma_k = 1$

& ii. at $e = e_{min}$, $\gamma_k = \gamma_{kmax}$

Again let's assume that at $e = e_{max}$ i.e. when $\gamma_k = 1$, the structure of the sample almost resembles to perfectly flocculated

i.e. each particle is perpendicular to the adjoining particle and for this state of particle orientation, let's assign a value of 1 to degree of parallelism . Similarly when $e = e_{min}$ i.e. when $\gamma_k = \gamma_{kmax}$, almost all the particles will be parallel to each other and degree of parallelism will have the maximum value of say 100. Therefore boundary conditions (i) and (ii) becomes

i. at $e = e_{max}$, $r_k = 1$ and $\eta = 1$

ii. at $e = e_{min}$, $r_k = r_{kmax}$ and $\eta = 100$

Putting 1st boundary condition in equation (6.3)

$$1 = A \frac{1}{e_{max}}$$

$$\text{or} \qquad A = e_{max} \qquad \ldots\ldots\ldots\ldots \quad 6.4$$

Equation (6.3) becomes

$$\gamma_k = \frac{e_{max}}{e} \eta^{\alpha} \qquad \ldots\ldots\ldots \quad 6.5$$

Putting 2nd boundary condition in above equation.

$$\gamma_{kmax} = \frac{e_{max}}{e_{min}} 100^{\alpha}$$

$$\text{or} \quad 100^{\alpha} = \frac{e_{min}}{e_{max}} r_{k\,max}$$

$$\text{or} \quad \log 100^{\alpha} = \log_{10} \frac{r_{kmax}\, e_{min}}{e_{max}}$$

$$\text{or} \quad \alpha \log 100 = \log_{10} \frac{r_{kmax}\, e_{min}}{e_{max}}$$

$$\text{or} \quad 2\alpha = \log_{10} \frac{r_{kmax}\, e_{min}}{e_{max}}$$

$$\text{or} \quad \alpha = \log_{10} \sqrt{\frac{r_{kmax}\, e_{min}}{e_{max}}} \qquad \ldots\ldots\ldots\ldots \quad 6.6$$

Now, e_{max}, e_{min} and r_{kmax} are constants for a particular soil and which can be determined in the laboratory. Therefore α and A are constants depending on soil type.

From equation (6.5)

$$r_k e = e_{max} \eta^{\alpha} \quad \text{or} \quad \eta = \left(\frac{r_k e}{e_{max}}\right)^{\frac{1}{\alpha}} \quad \ldots\ldots \quad 6.7$$

$$\text{for } e_{min} < e < e_{max}.$$

Equation (6.7) suggests a method to find the degree of parallelism on the proposed structural scale at any void ratio e, for a particular α which is a constant depending on type of soil.

6.3 EXAMPLE ILLUSTRATING THE PROPOSED HYPOTHESIS:

An example of structural scale as found out from equation (6.7) for kaolinite + 5% Na oxalate soil type is given here.

For this soil

$$\left.\begin{aligned} e_{max} &= 0.9 \\ e_{min} &= 0.69 \\ r_{kmax} &= 1.6 \end{aligned}\right\} \text{Found by extrapolating the curve in figure 14.}$$

$$\alpha = \log_{10}\sqrt{\frac{r_{kmax} e_{min}}{e_{max}}} = \log_{10}\sqrt{\frac{1.6 \times 0.69}{0.9}}$$

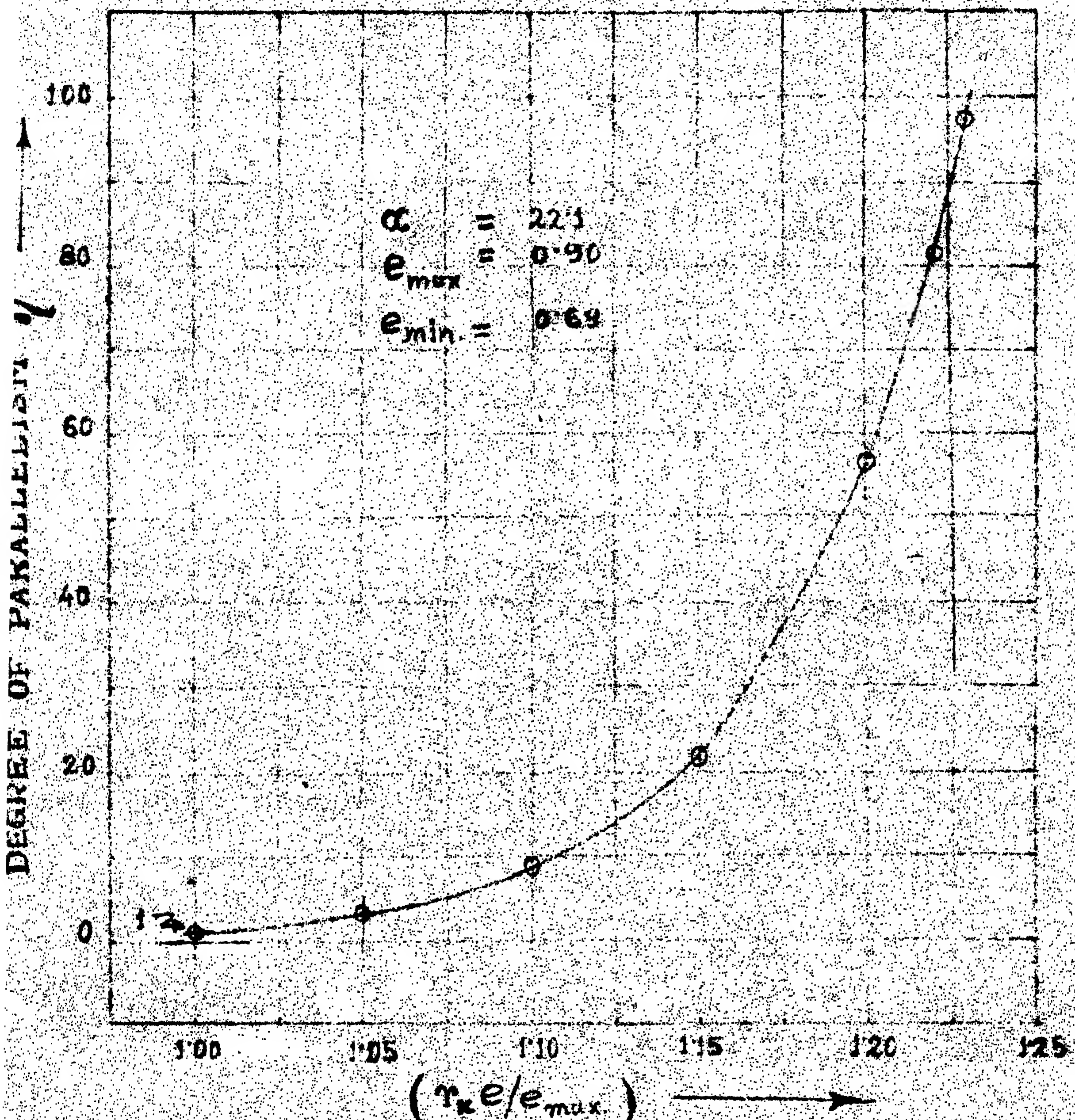
$$= \log_{10}\sqrt{1.227} = \log_{10} 1.11$$

or $\alpha = 0.0453$

$$\frac{1}{\alpha} = 22.1$$

The equation (6.7) for this particular soil becomes

FIG. 18



DEGREE OF PARALLELESM 'η' Vs. $r_x e/e_{max}$ FOR KAOLINITE WITH 5% DISPERSING AGENT

$$\eta = \left(\frac{r_k e}{e_{max}}\right)^{22.1} \quad \ldots\ldots\ldots\ldots\ldots \quad 6.8$$

A graphical plot of this equation is shown fig. (18). which gives η for any e, provided we know r_k at that e.

So, it can be concluded that permeability ratio r_k do give an indication about the structure or degree of parallelism at that void ratio and the hypothesis forwarded gives an apportunity to assign a value within the structural scale.

Further work is necessary to verify the validity of the above hypothesis for several types of soils and over larger range of void ratios.

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CHAPTER 7

A STUDY OF SHAPE FACTOR FOR SAND

7.1 INTRODUCTION:

The influence of shape of flow path on the permeability of sand is a well known phenomena. In a fluid, undergoing laminar flow through a circular pipe, the quantity of flow is given by (3)

$$Q_{cir} = \frac{1}{2} \frac{\gamma R_H^2}{\mu} i\, a \quad \ldots\ldots\ldots\ldots \quad 7.1$$

And for flow through parallel plates

$$Q_{pl} = \frac{1}{3} \frac{\gamma R_H^2}{\mu} i\, a \quad \ldots\ldots\ldots\ldots \quad 7.2$$

where R_H = hydraulic radius

i = hydraulic gradient

a = Area through which flow in occuring.

Using the concept of hydraulic radius in case of soils (6) the equation for quantity of flow through soils becomes

$$Q = (D_s^2 \frac{\gamma}{u} \frac{e^3}{1+e} C)\, i\, a \quad \ldots\ldots\ldots\ldots \quad 7.3$$

comparing this equation with the Darcy's equation for laminar flow through soil

$$Q = K\, i\, A \quad \ldots\ldots\ldots\ldots \quad 7.4$$

we find

$$K = CD_s^2 \frac{\gamma}{u} \frac{e^3}{1+e} \quad \ldots\ldots\ldots\ldots \quad 7.5$$

where

K = Darcy Coefficient of permeability

D_s = some effective particle diameter

γ = Unit weight of permeant

μ = Viscosity of permeant

e = Void ratio of the sample

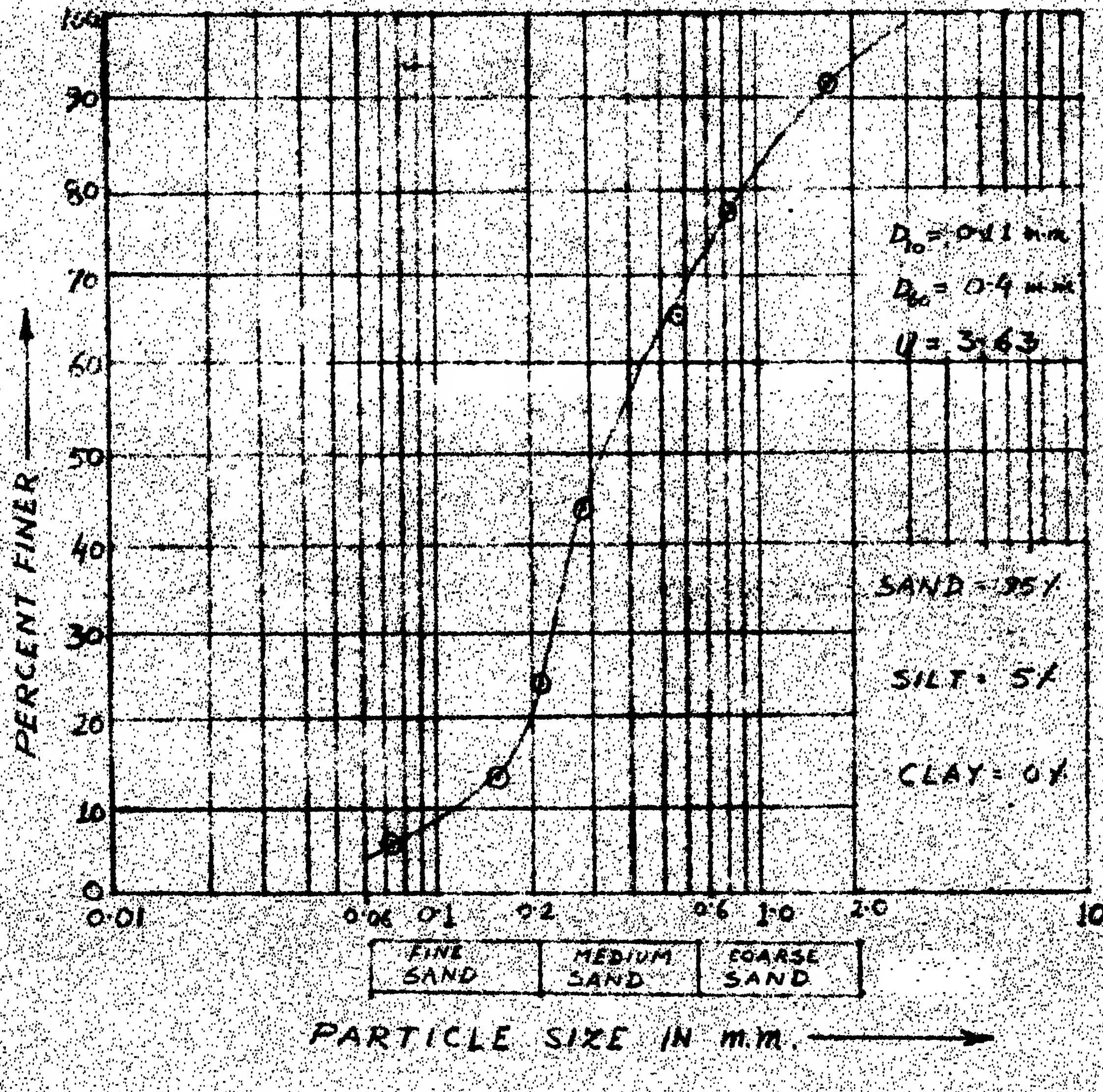
C = A factor known as composite shape factor depending on shape of flow path and hence depends on void ratio for a particular type of grain shape.

As seen from equation 7.1 and 7.2 the shape factor (C) for circular flow path is $\frac{1}{2}$ and that of for flow between two parallel plates is 1/3 but the flow path through a soil is extremely complex in nature and hence it will be extremely difficult, if not impossible, to evaluate the numerical value of this shape factor by any theoretical means. So an experimental study of this shape factor using equation 7.5 is tried and is presented here. For this study, two distinct types of sand of known average diameters were used and shape factors, were evaluated and factors affecting it were studied.

7.2 SAMPLES TESTED AND PROCEDURE FOLLOWED:

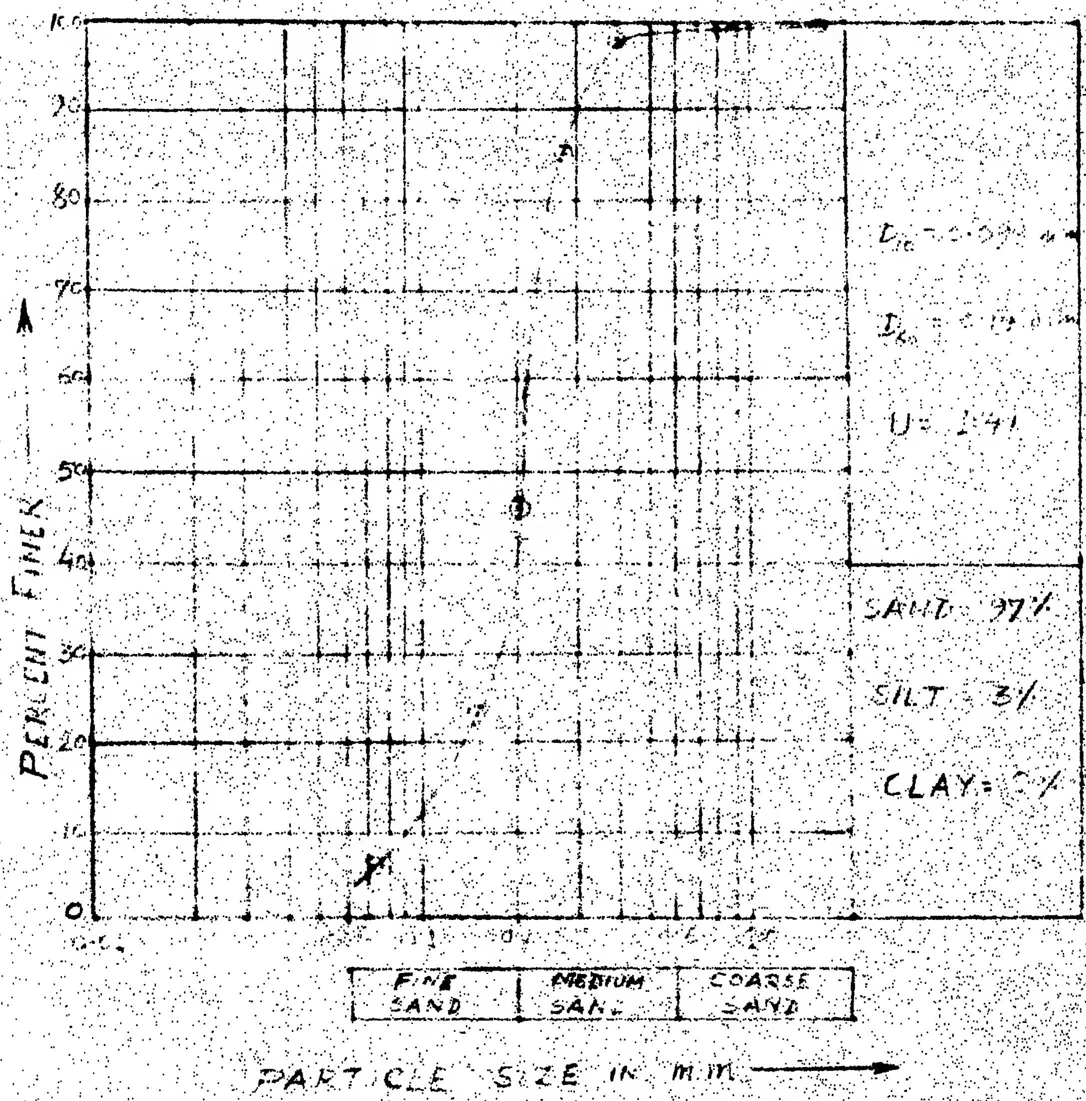
Two distinct different types of sand one is known as Kalpi sand and the other as Ganges sand were collected from local deposits and the gradation curve of which are shown in fig. 19 and 20. For the tests, the samples were sieved and grouped according to the grain size. For Kalpi sand, the first group consists of sand passing B.S. 18 and retaining on B.S. 36, second group consists

FIG. 19



GRAIN SIZE CURVE FOR KALPI SAND.

FIG 20



GRAIN SIZE ANALYSIS OF GANGES SAND

of sand passing B.S. 36 and retaining on B.S. 52 and the 3rd group, passing B.S, 52 and retaining B.S. 72. Similarly for ganges sand, the groups are (1) passing B.S. 36 and retaining B.S. 52, (2) passing B.S. 52 retaining B.S. 72 and (3) passing 72 retaining B.S. 100. The average diameter of the sand particles are calculated as the arithmatic average of the opening of the respective sieve numbers and is shown in table 10. A microscopic study of the different samples tested were made and the results along with the magnified photos of the sand grains are shown in photoplate no. 3 and 4.

TABLE 10

AVERAGE GRAIN DIAMETERS

Passing Sieve No.	Retaining on	Average diameter in m.m.
B.S. 18	B.S. 36	$\frac{0.853+0.422}{2} = 0.637$
B.S. 36	B.S. 52	$\frac{0.422+0.295}{2} = 0.358$
B.S. 52	B.S. 72	$\frac{0.295 + 0.211}{2} = 0.253$
B.S. 72	B.S. 100	$\frac{0.211 + 0.152}{2} = 0.181$

Constant head vertical permeability tests were conducted on sands of three different average diameters as mentioned at three different void ratios - minimum and maximum possible and one intermediate. The height of the sand column was measured accurately each time to determine the void ratio. The looset void ratio was obtained by pouring the sand through a funnel and the maximum by vibrating the permeameter for sufficiently long time (approximately 5 minuates). Temperature correction was

MICROSCOPIC VIEW OF KALPI SAND

Passing B.S. 18 retained on B.S. 36

d_{av} = 0.637 mm

shown 23 times magnified

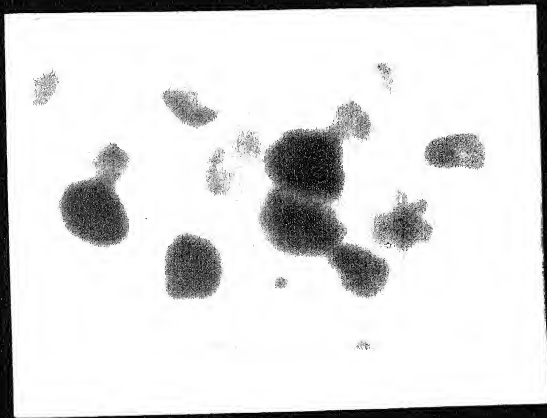
Subrounded, grains coated with iron oxide.

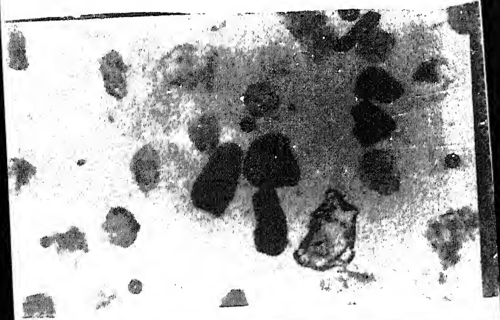
sing B.S. 36 retained on B.S. 52

d_{av} = 0.358 mm

Shown 22 times magnified

Relatively angular and comparatively transparent





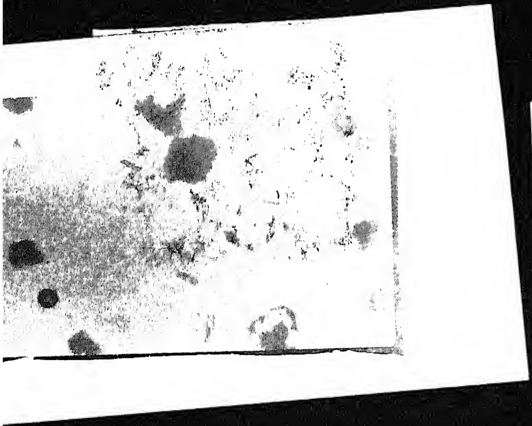
Passing B.S. 52 retained on B.S. 72

d_{av} = 0.253 mm

Shown 24 times magnified

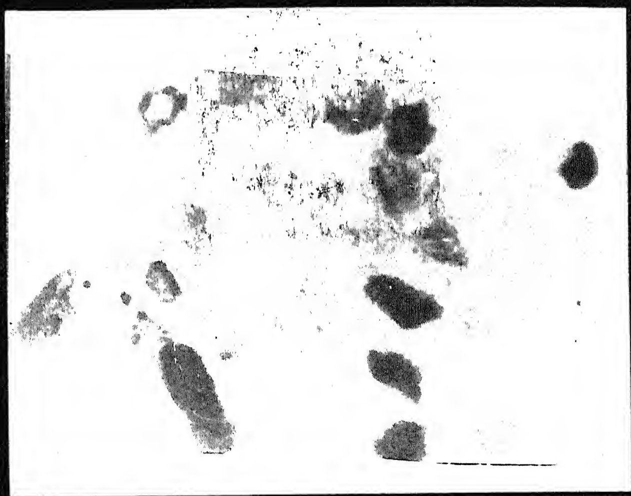
Subrounded, Zarcons are present in addition to quartz.

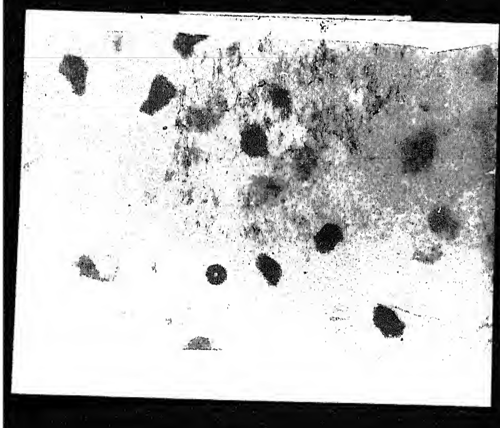
MICROSCOPIC VIEW OF GANGES SAND



Passing B.S. 36 retained on B.S.52
d_{av} = 0.358 mm
Shown 17 times magnified
Very angular, differs from Kalpi Sand of same size and contains mica

sing B.S. 52 retained on B.S. 72
= 0.253 mm
wn 28 times magnified
ılar, Zarcons and quartz are present





Passing B.S. 72 retained on B.S. 100
d_{av} = 0.181 mm
Shown 17 times magnified
Angular, quartz are coated with iron oxide.

applied to each reading as explained in section 4.6. Other precautions as discussed in section 4.7 were also taken.

A preliminary test on the pure Kalpi and Ganges sand showed that the flow through it was not perfectly Laminar ('v' superficial velocity was not directly proportional to hydraulic gradient applied). So to ensure perfect laminar flow 5% kaolinite (by weight) was added & thoroughly mixed with the sands and then the tests were conducted as discussed in previous para.

7.3 VERTICAL PERMEABILITY RESULTS AND DISCUSSION:

As discussed in section 7.2 the vertical permeability tests were conducted and the results for Kalpi sand is shown in table 11 and that of Ganges sand is shown in table 12. From the measured data a plot of k_v at 20°c vs. $\frac{e^3}{1+e}$ is obtained for both kalpi and ganges sand for different average diameters and are shown in fig. 21 and 22 . The plot shows a non linear variation between k_v vs. $\frac{e^3}{1+e}$ indicating that the shape constant 'C' depends on void ratio and grain size. Figure 21 and 22 also shows that at the same void ratio permeability is more for samples with bigger diameter grains. Using fig. 21 and 22 a plot has been obtained between permeability k_v and average grain diameter 'd' for different void ratios as shown in fig. 23 and 24. The fig. shows that at a particular void ratio, the increase in permeability with grain diameter is sharp at smaller diameters but the increment rate is very small at larger grain diameters.

TABLE 11

VERTICAL PERMEABILITY RESULTS OF KALPI SAND

Average grain diam. in m.m	e	h in cm	Q in cc	t in sec	L in cm	$k_v \times 10^{-4}$ in cm/sec	T in $^o c$	$\frac{\mu_T}{\mu_{20}}$	$k_v \times 10^{-4}$ at $20^o c$ in cm/sec.
0.637	0.905	111.6	60	60	8.5	9.52	35	0.715	6.8
			61	60					
	0.84	111.6	42	60	8.2	6.44	34	0.73	4.7
			42	60					
	0.672	111.6	18	60	7.45	2.9	33	0.745	2.16
			18	60					
0.358	0.94	111.6	55	60	9.6	8.95	29	0.812	7.26
			55	60					
	0.818	111.6	28	60	9.0	4.7	29	0.812	3.82
			28	60					
	0.738	111.6	20	60	8.6	3.21	28.5	0.82	2.63
			20	60					
0.253	0.99	111.6	70	60	6.9	9.02	32	0.76	6.85
			70	60					
	0.79	111.6	33	60	6.2	3.81	32	0.76	2.89
			33	60					
	0.73	111.6	21	60	6.0	2.35	29	0.812	1.905
			21	60					

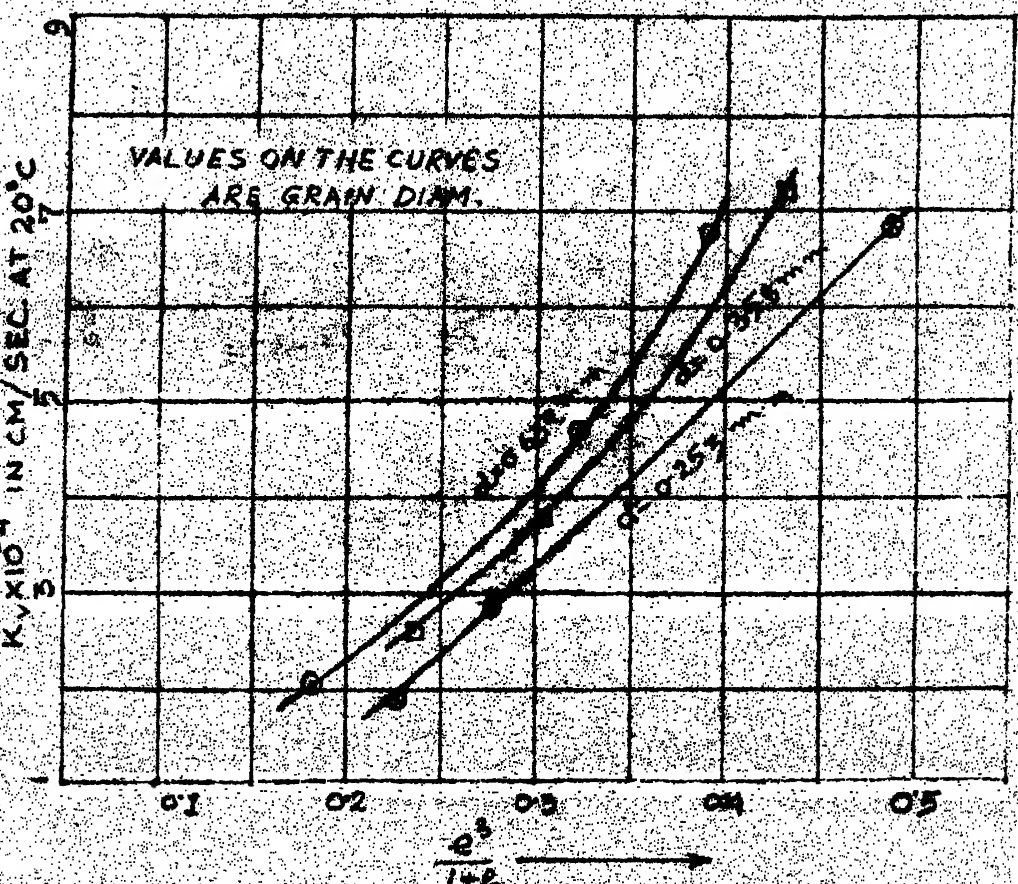
TABLE 12

VERTICAL PERMEABILITY RESULTS OF GANGES SAND

66

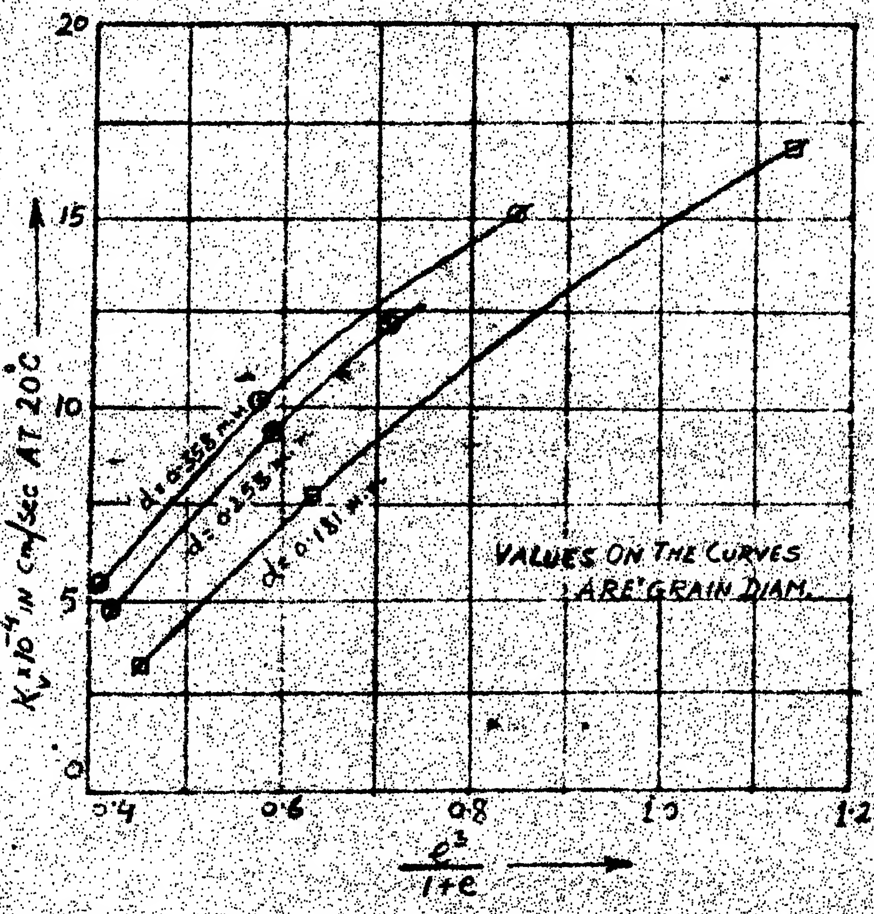
Average Grain Diam. in m.m.	e	h in cm	Q in cc	t in sec	L in c.m.	$k_v \times 10^{-4}$ in cm/sec	T in °c	μ_T / μ_{20}	$k_v \times 10^{-4}$ at 20° in cm/sec.
			96	60					
	1.236	111.6			10.1	18.1	27.5	0.835	15.1
			96	60					
			73	60					
	1.05	111.6			9.7	12.4	27.5	0.835	10.5
0.358			73	60					
			40	60					
	0.922	111.6			9.1	6.8	27.5	0.835	5.68
			40	60					
			88	60					
	1.14	111.6			9.15	15.05	28	0.829	12.48
			88	60					
			73	60					
0.253	1.06	111.6			8.8	12.0	28.5	0.82	9.85
			73	60					
			40	60					
	0.93	111.6			8.25	6.0	28.5	0.82	4.83
			40	60					
			110	60					
	1.39	111.6			10.9	22.4	32.5	0.752	16.95
			110	60					
			51	60					
0.181	1.11	111.6			9.65	9.2	27.5	0.835	7.68
			51	60					
			24	60					
	0.36	111.6			8.95	3.97	27.5	0.835	3.31
			24	60					

FIG - 21



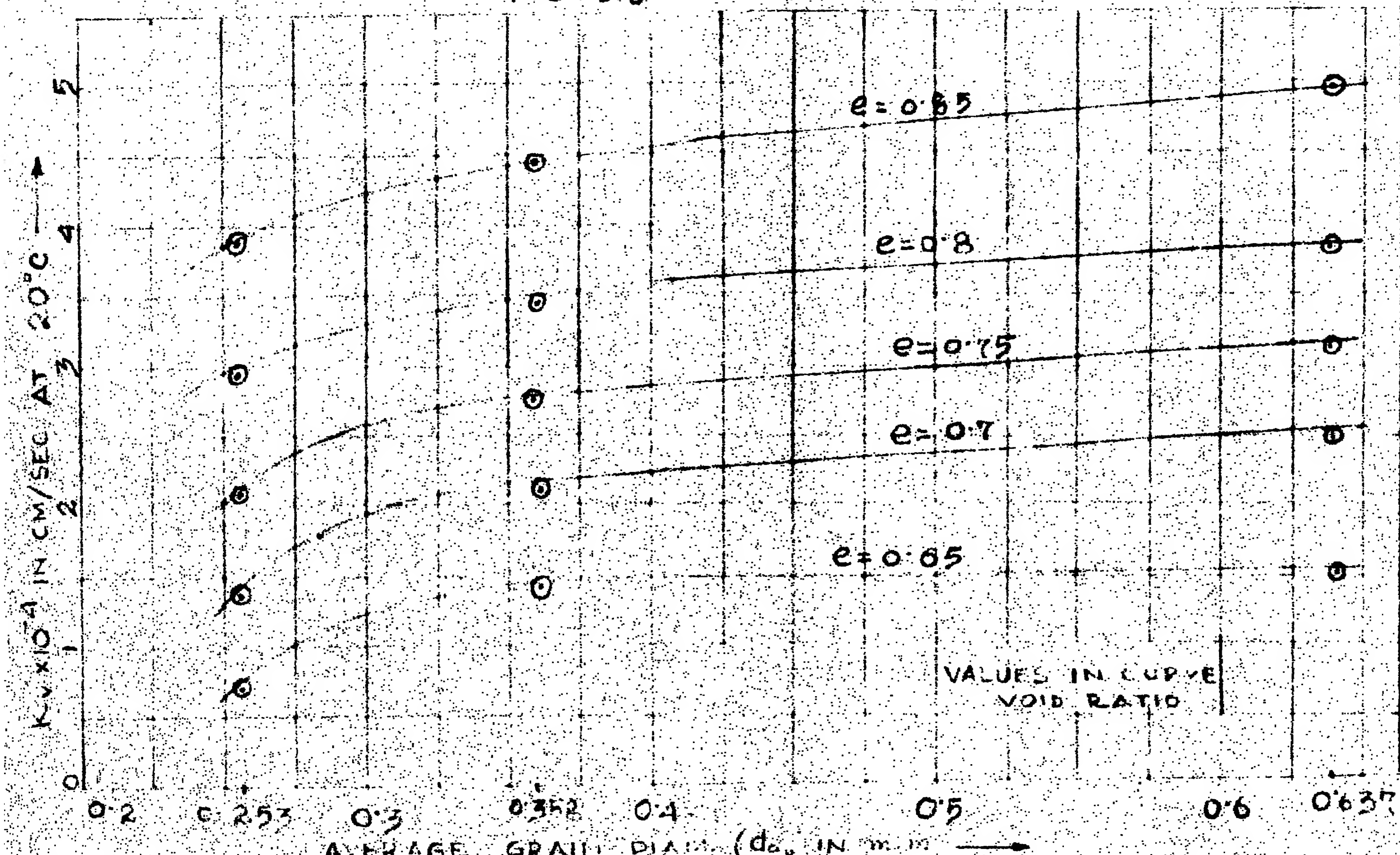
VARIATION OF K_v OF KALPISAND WITH $\frac{e^3}{1+e}$ AT DIFF. GRAIN DIA.

FIG. 22



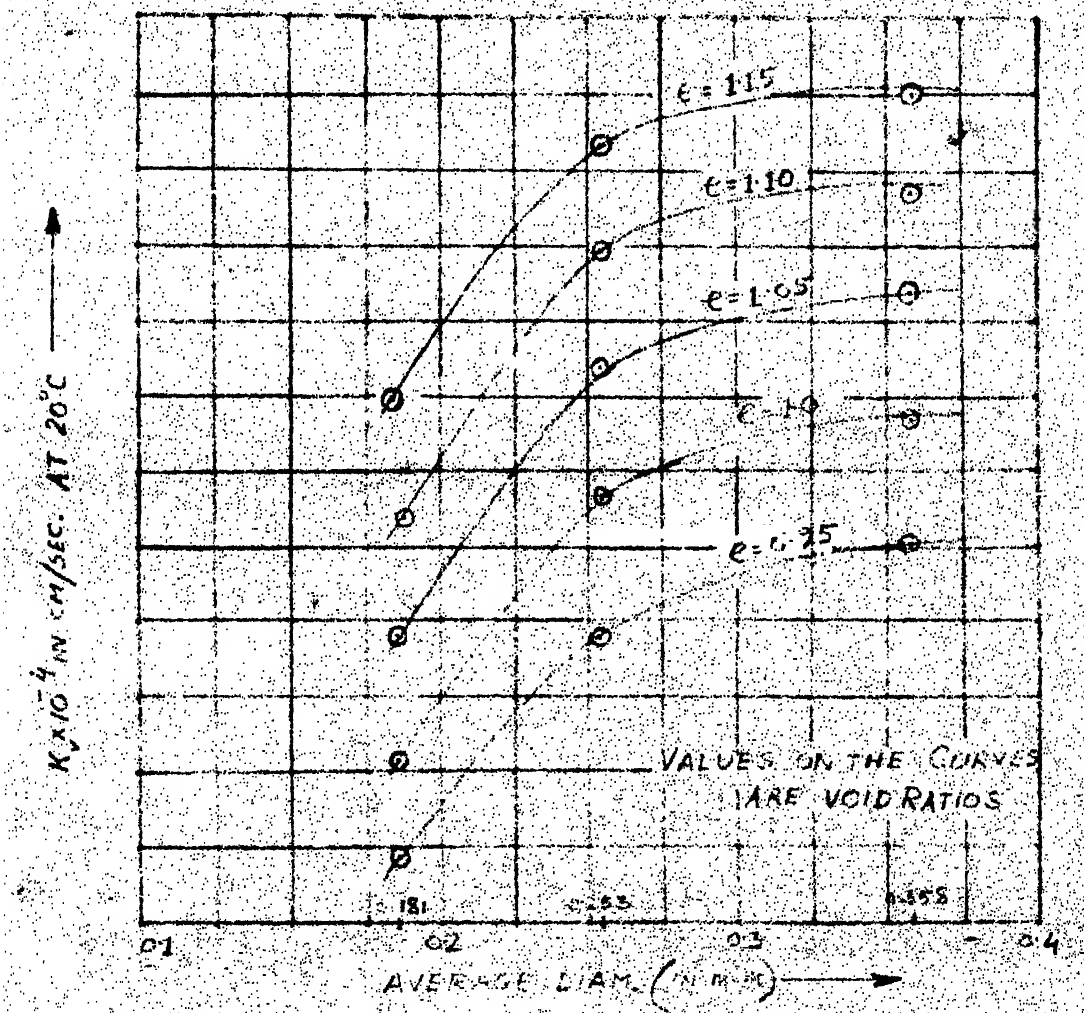
VARIATION OF K_v WITH $\frac{e^3}{1+e}$ AT DIFF. GRAIN DIAM.
OF GANGES SAND.

FIG-13



VARIATION OF K_v OF KA. PI..AD WITH d_{av} AT DIFF. VOID RATIO e

FIG. 24



VARIATION OF K_v WITH GRAIN DIAM. AT DIFF. VOID RATIOS
OF
GANGES SAND.

7.4 EVALUATION AND STUDY OF SHAPE FACTORS:

Shape factors at different void ratios and grain diameters are calculated from the test results and with the help of equation 7.5. From Equation 7.5

$$C = \frac{K \frac{u}{\gamma} \frac{1}{D_s^2}}{\frac{e^3}{1+e}} \quad . \; . \; . \; . \; . \; . \; . \; . \; . \qquad 7.6$$

All the permeability values, used in equation (7.6) to evaluate C, are at $20^\circ c$

$$u = \text{viscosity of water at } 20^\circ c = \frac{0.01009}{980} \text{gm sec/cm}^2$$
$$= 1.029 \times 10^{-5} \text{ gm sec/cm}^2$$

and γ = unit wt. of water at $20^\circ c$ = 0.99823 gm/cm^3

$$\left(\frac{u}{\gamma}\right) \text{ at } 20^\circ c = \frac{1.029 \times 10^{-5}}{0.99823} \text{ cm sec.} \simeq 102.9 \times 10^{-7} \text{cm sec.}$$

Equation (7.6) becomes

$$C = 102.9 \times 10^{-7} \frac{\frac{K}{D_s^2}}{\frac{e^3}{1+e}} \quad . \; . \; . \; . \qquad 7.7$$

The value of D_s is taken as the average diam. of the grains as shown in table 10.

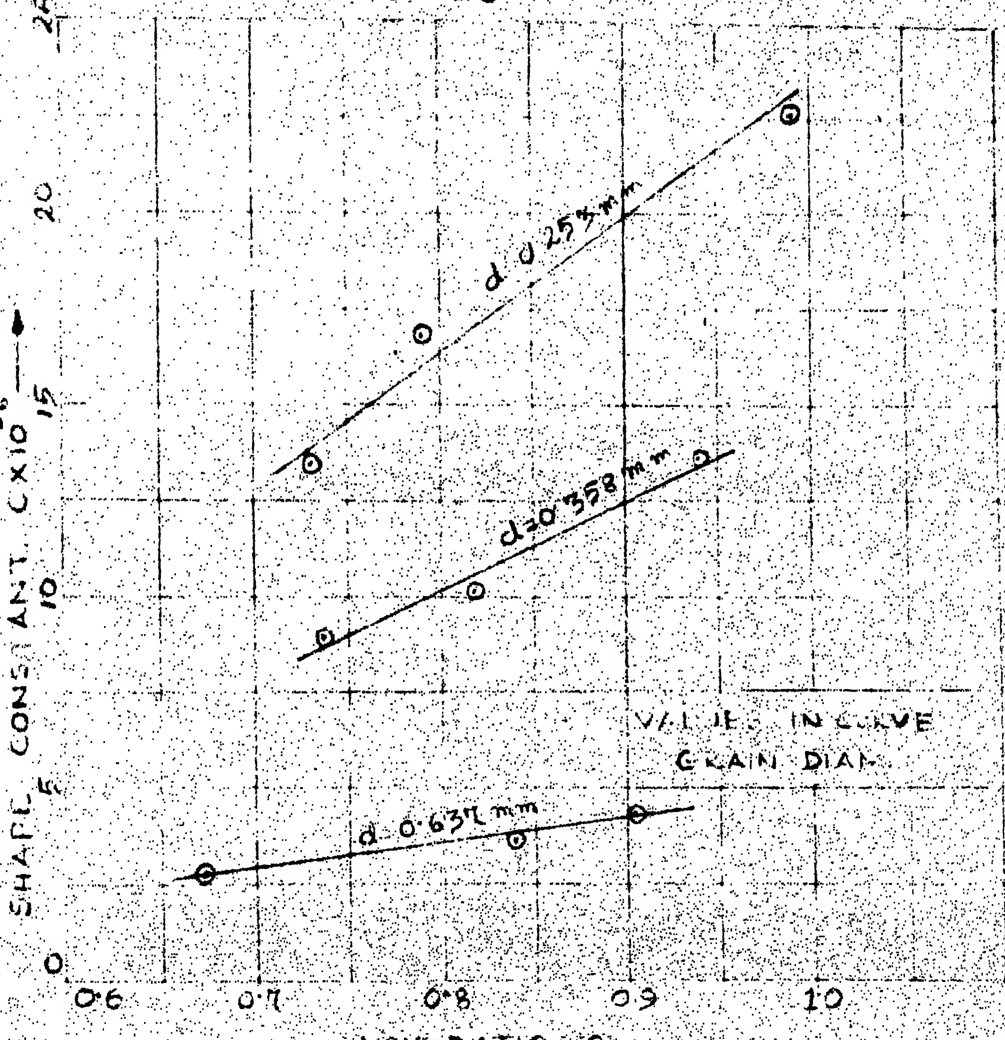
Fitting the values of K for particular D_s and e, from table 11 and 12, shape factor values are calculated and shown in table 13. From this table a plot of shape factor C vs. void ratio e for different values of average diam. d are plotted and shown in fig. 25 and 26. From the plot, it is seen that for kalpi sand, the value of shape factor 'C' increases linearly with void ratio, for a particular grain diameter within the void ratio range of 0.7 to 0.95. The rate of increase of C with e is comparitively smaller for larger diameters. The overall trend for ganges sand as seen in fig. 26

TABLE 13

EVALUATION OF SHAPE FACTOR

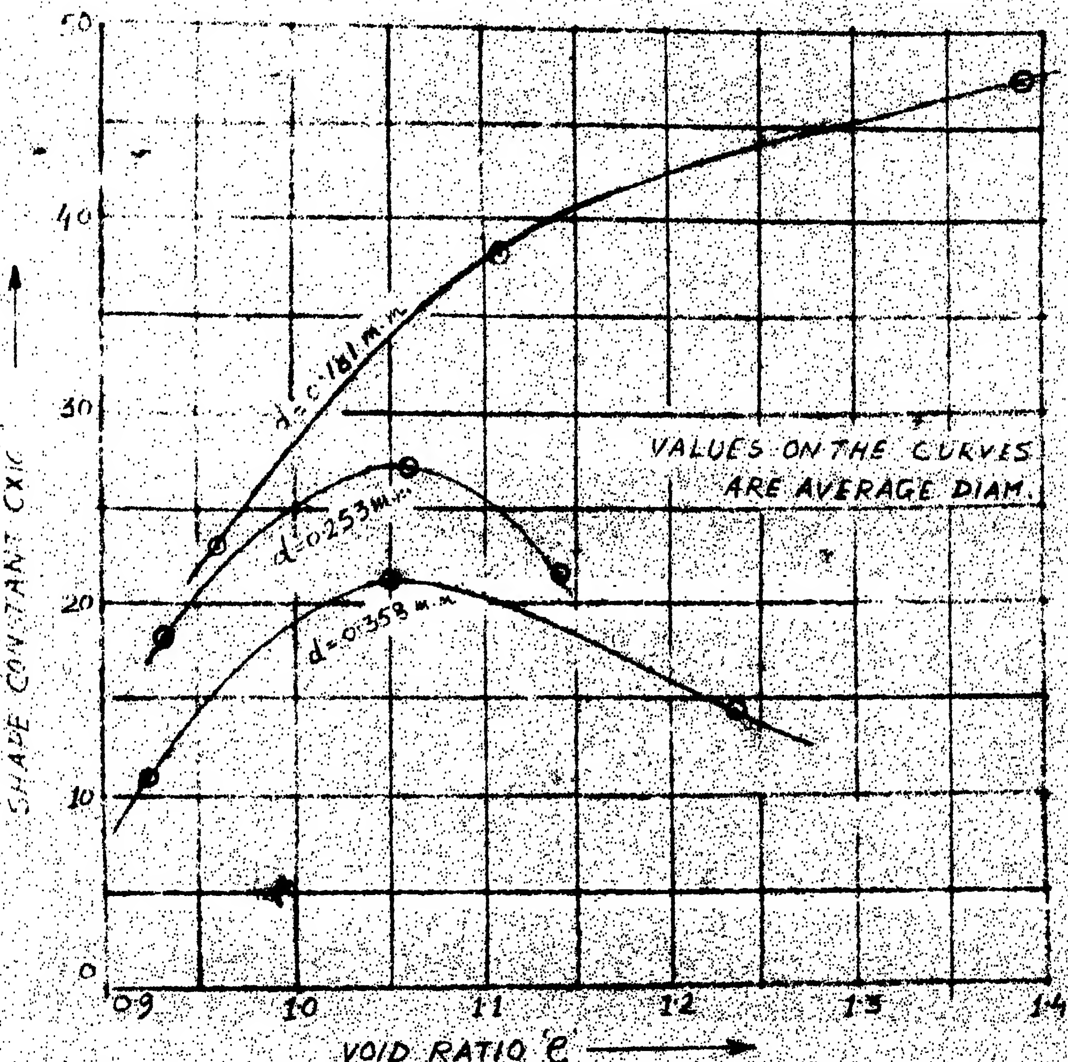
Type of sand	D_s = Average Diam. in m.m.	e	$\frac{e^3}{1+e}$	$Kx10^{-4}$ in cm/sec	$Cx\ 10^{-6}$
		1.236	0.85	15.1	14.2
	0.358	1.05	0.566	10.5	21.4
		0.922	0.406	5.6	11.0
Ganges		1.14	0.702	12.48	28.6
sand	0.253	1.06	0.582	9.85	27.2
		0.93	0.42	4.83	18.5
		1.39	1.12	16.95	47.5
	0.181	1.11	0.63	7.68	38.4
		0.96	0.452	3.31	23
		0.905	0.389	6.8	4.34
	0.637	0.84	0.324	4.7	3.6
		0.672	0.182	2.16	2.94
		0.94	0.428	7.26	13.6
Kalpi sand	0.358	0.818	0.303	3.82	10.1
		0.738	0.234	2.63	9.0
		0.99	0.488	6.85	22.6
	0.253	0.79	0.277	2.89	16.8
		0.73	0.226	1.905	13.5

FIG. -25



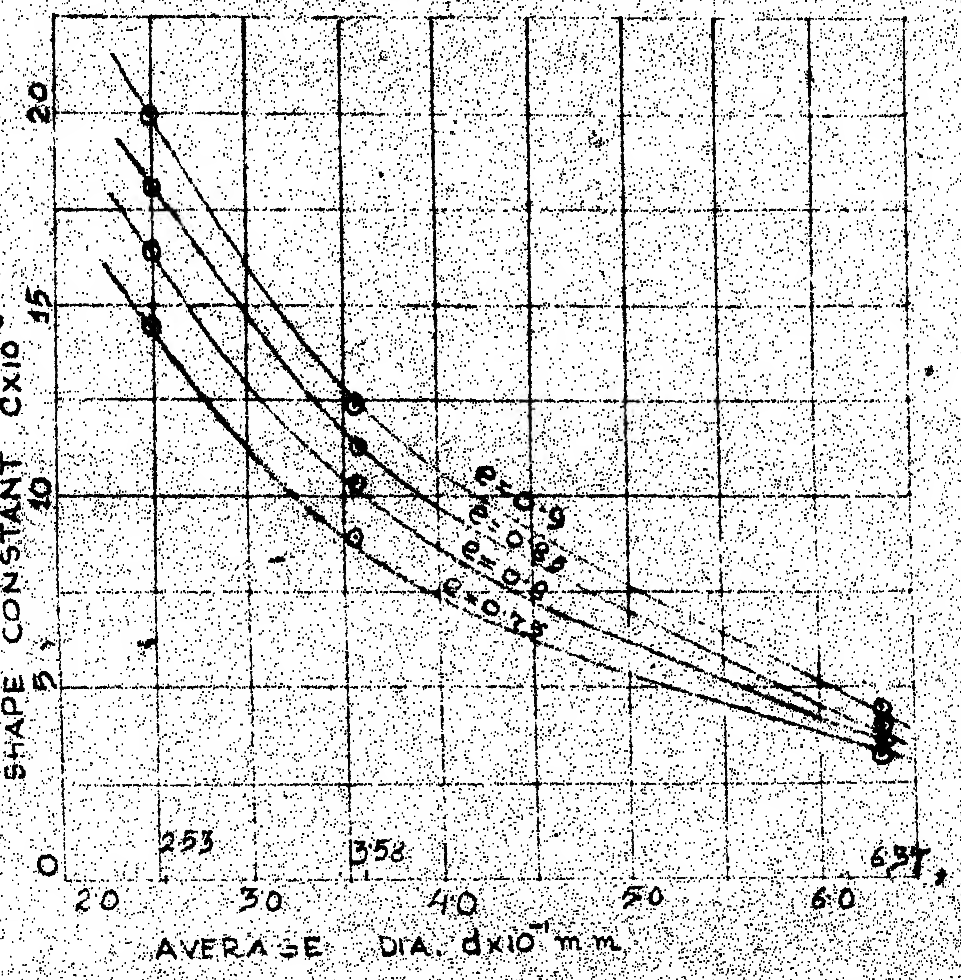
VARIATION OF SHAPE CONSTANT
OF KALPI SAND WITH 'e' AND
AVERAGE DIA d

FIG. 26



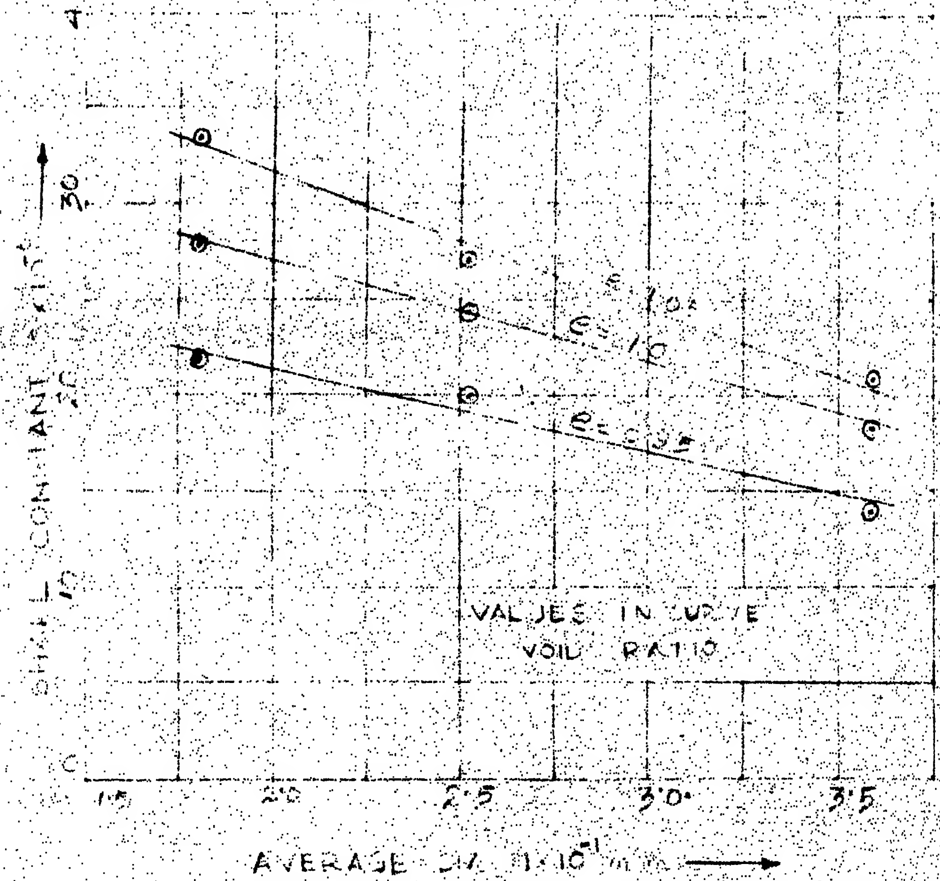
VARIATION OF SHAPE CONSTANT 'C' OF GANGES SAND WITH 'e' AND AVERAGE DIA. 'd'

FIG - 27



VARIATION OF 'C' WITH 'd' AT DIFF. 'e' OF KALPI SAND

FIG - 28



VARIATION OF SHAPE CONSTANT C
WITH d AT DIFF. e OF GANGES SAND

is a bit different. Within the void ratio range of 0.9 to 1.2, for the average diameter of 0.253 m.m. and 0.358 m.m. the shape factor C increases upto the void ratio of 1.05 and then gradualy decreases, but for the average diameter of 0.818 m.m. the decreasing trend is missing upto the void ratio of approximately 1.4.

From the fig. 25 and 26, values of C at different diameters at some selective void ratios are calculated and plotted and shown in fig. 27 and 28. As seen from these plots at all void ratios shape factor C tends to attain one unique value at large diameters. Therefore we can say that only for samples with larger grain sizes, shape factor 'C' is independent of void ratio.

7.5 CONCLUSIONS:

From the above discussions and test results the following conclusions can be drawn.

1. Shape factor 'C' is a function of void ratio and grain size and is not truely a constant as was assumed in equation 7.5.
2. For larger grain size shape factor C assumes an unique value irrespective of void ratios.
3. The rate of change of shape factor with void ratio is very less for large grain size.
4. For ganges sand, shape factor increases and then decreases after a certain void ratio but for kalpi sand the decreasing tendency is absent within the void ratio range tested.

5. The rate of increase of permeability with grain diameter at all void ratios is higher at smaller grain diameters whereas it is comparitively lower at larger grain diameters.

6. The coefficient of permeability K is not linearly proportional to $\frac{e^3}{1+e}$ for kalpi and ganges sand tested.

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CHAPTER 8

DEPTH DEPENDENT ANISOTROPIC PERMEABILITY

8.1 INTRODUCTION:

The void ratio of soils depend upon the consolidation pressure and hence it is expected that within a given formation the void ratios generally decrease with depth. This phenomenon indicates non homeogeneity and anisotropy. In problems involving flow through such formations the hydraulic properties will be anisotropic and depth dependent. Assuming a linear decrease of void ratio with depth, analytical solution for the vertical and horizontal permeabilities and the permeability ratio are presented.

Mansur and Dietrich (7) and Jackobson (8) reported some results of field tests to determine permeability ratio at two different sites. Kenney (9) gave an analytical expression between permeability ratio and the ratio of permeabilities at two different depths by assuming a linear variation of permeability with depth for a repeatedly layered soils. Siraskar and Patel (10), following the same line but assuming a parabolic distribution of permeability with depth found out an expression between the above ratios.

The variation of permeability with depth is primarily caused by the variation of void ratio . Besides, for clays the linear relation between void ratio and logarithm of permeability is an experimentally established fact (3). Hence, it is more

realistic to utilize this relationship along with the assumption that the void ratio varies with depth. Different solutions can be obtained for different depth - void ratio relationships. A linear decrease of void ratio with depth is assumed here. The solutions presented here relate the permeability ratio to the average void ratios over a depth and is convinient for visualisation of the depth dependent hydraulic properties.

8.2 DERIVATION OF EQUATIONS:

Let the thickness of the aniosotropic and non homogeneous clay strata be 'H'. The void ratio distribution along the depth is assumed to be linear as shown in fig 29. The equation relating void ratio 'e', with depth 'h' can be written as

$$e = e_o - t\,h \quad \ldots\ldots\ldots\ldots \quad 8.1$$

where e_o = void ratio at h = o

The equation relating permeability K and the void ratio can be written as (see fig. 30).

$$e = m \log K + C$$

Let $C = m \log n$, where n is a constant

Hence

$$e = m \log K + m \log n$$

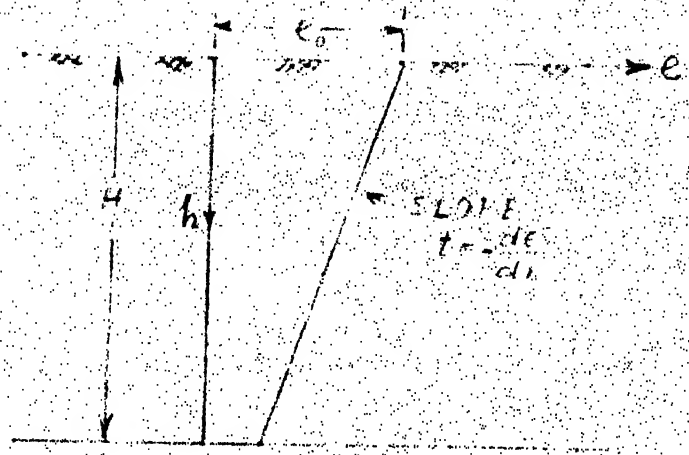
$$= m \log n K$$

or

$$K = \frac{1}{n}\left[\epsilon^{\frac{e}{m}}\right] \quad \ldots\ldots\ldots \quad 8.2$$

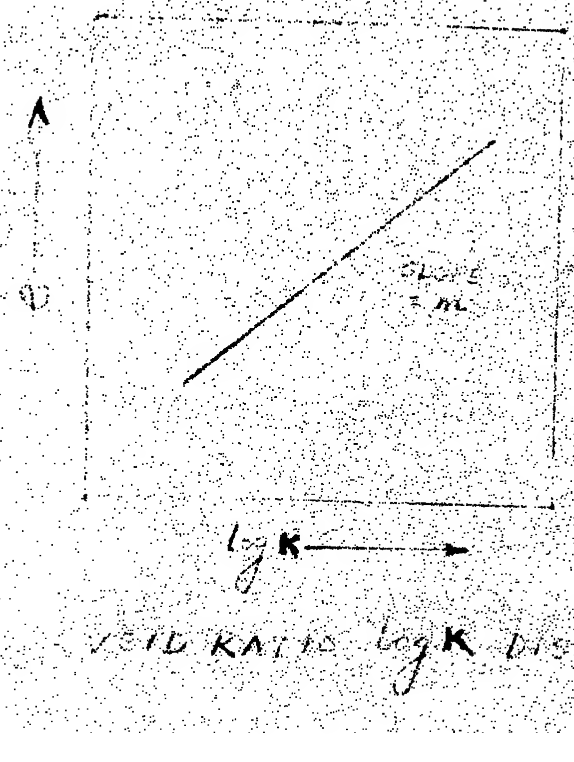
Substituting for e from equation 8.1 into equation 8.2, the expression for the permeability K at any depth 'h' is given by

FIG 29



ASSUMED VOID RATIO DISTRIBUTION
ALONG DEPTH

FIG 30



VOID RATIO $\log K$ DISTRIBUTION

$$K = \frac{1}{n} \epsilon^{\frac{e_o - t h}{m}} = \frac{1}{n} \epsilon^{\frac{e_o}{m}} \epsilon^{-\frac{t}{m} h}$$

$$\text{or} \quad K = \gamma \epsilon^{-\alpha h} \quad \ldots\ldots\ldots\ldots \quad 8.3$$

where $\alpha = \frac{t}{m}$ and $\gamma = \frac{1}{n} \epsilon^{\frac{e_o}{m}}$ signifying the value of permeability at h = o

Now, if we consider that the whole clay stratum of thickness H is composed of small layers of thickness 'dh' with permeability K, then the average horizontal permeability of the whole stratum is given by

$$K_x = \frac{1}{H} \int_0^H K dh$$

Putting equation 8.3 in this expression, we get

$$K_x = \frac{1}{H} \int_0^H \gamma \epsilon^{-\alpha h} dh = - \frac{\gamma}{\alpha H} \left[\epsilon^{-\alpha H} - 1 \right]$$

$$\text{or} \quad K_x = \frac{\gamma}{\alpha H} \epsilon^{-\frac{\alpha H}{2}} \left[\epsilon^{+\frac{\alpha H}{2}} - \epsilon^{-\frac{\alpha H}{2}} \right]$$

$$\text{or} \quad K_x = \gamma \epsilon^{-\frac{\alpha H}{2}} \left[\frac{\operatorname{Sin} h \frac{\alpha H}{2}}{\frac{\alpha H}{2}} \right] \ldots\ldots\ldots \quad 8.4$$

Similarly, average vertical permeability K_Z of the whole stratum is given by

$$K_Z = \frac{H}{\int_0^H \frac{dh}{K}}$$

From equation (8.3), $K_Z = \dfrac{H}{\int_0^H \dfrac{dh}{\gamma \epsilon^{-\alpha h}}}$

or $K_Z = \dfrac{\gamma \alpha H}{\epsilon^{\alpha H} - 1}$

or $K_Z = \dfrac{\gamma \alpha H}{\epsilon^{\frac{\alpha H}{2}} \left[\epsilon^{\frac{\alpha H}{2}} - \epsilon^{-\frac{\alpha H}{2}} \right]}$

or $$K_Z = \gamma \epsilon^{-\frac{\alpha H}{2}} \left[\frac{\frac{\alpha H}{2}}{\operatorname{Sin} h \frac{\alpha H}{2}} \right] \quad \ldots\ldots \quad 8.5$$

From equation 8.4 and 8.5

$$\gamma_K = \frac{K_x}{K_Z} = \left[\frac{\operatorname{Sin} h \frac{\alpha H}{2}}{\frac{\alpha H}{2}} \right]^2 \ldots\ldots \quad 8.6$$

Now, from equation 8.1, average void ratio for the whole stratum 'H' is

$$e_{av} = \frac{1}{H} \int_0^H e \, dh = \frac{1}{H} \int_0^H (e_o - t\,h) \, dh$$

or $e_{av} = e_o - t \dfrac{H}{2}$

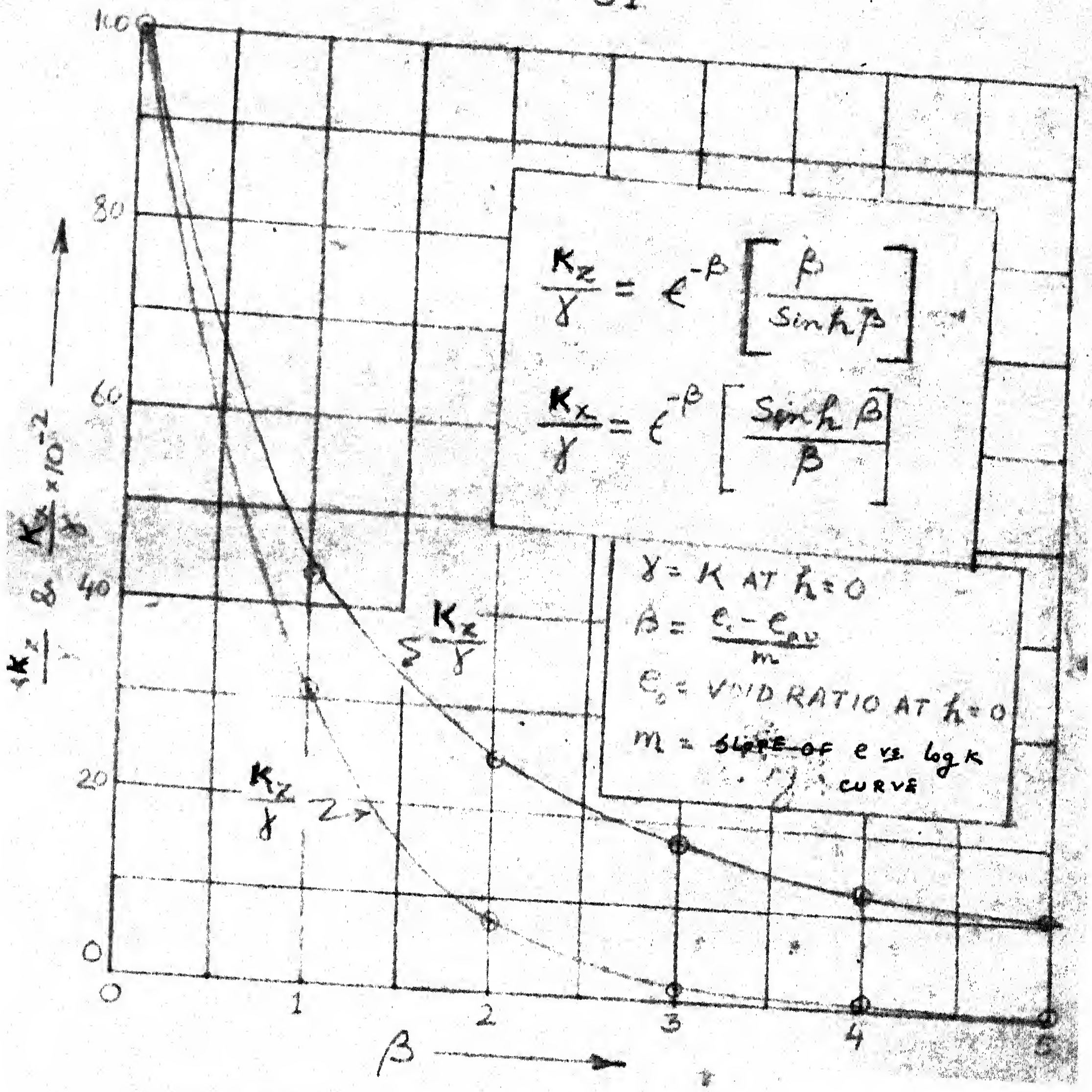
or $$H = \frac{2(e_o - e_{av})}{t} \quad \ldots\ldots\ldots \quad 8.7$$

From equation 8.7 and 8.4

$$K_x = \gamma \epsilon^{\frac{-\alpha(e_o - e_{av})}{t}} \left[\frac{\operatorname{Sin} h \frac{\alpha}{t} (e_o - e_{av})}{\frac{\alpha}{t} (e_o - e_{av})} \right]$$

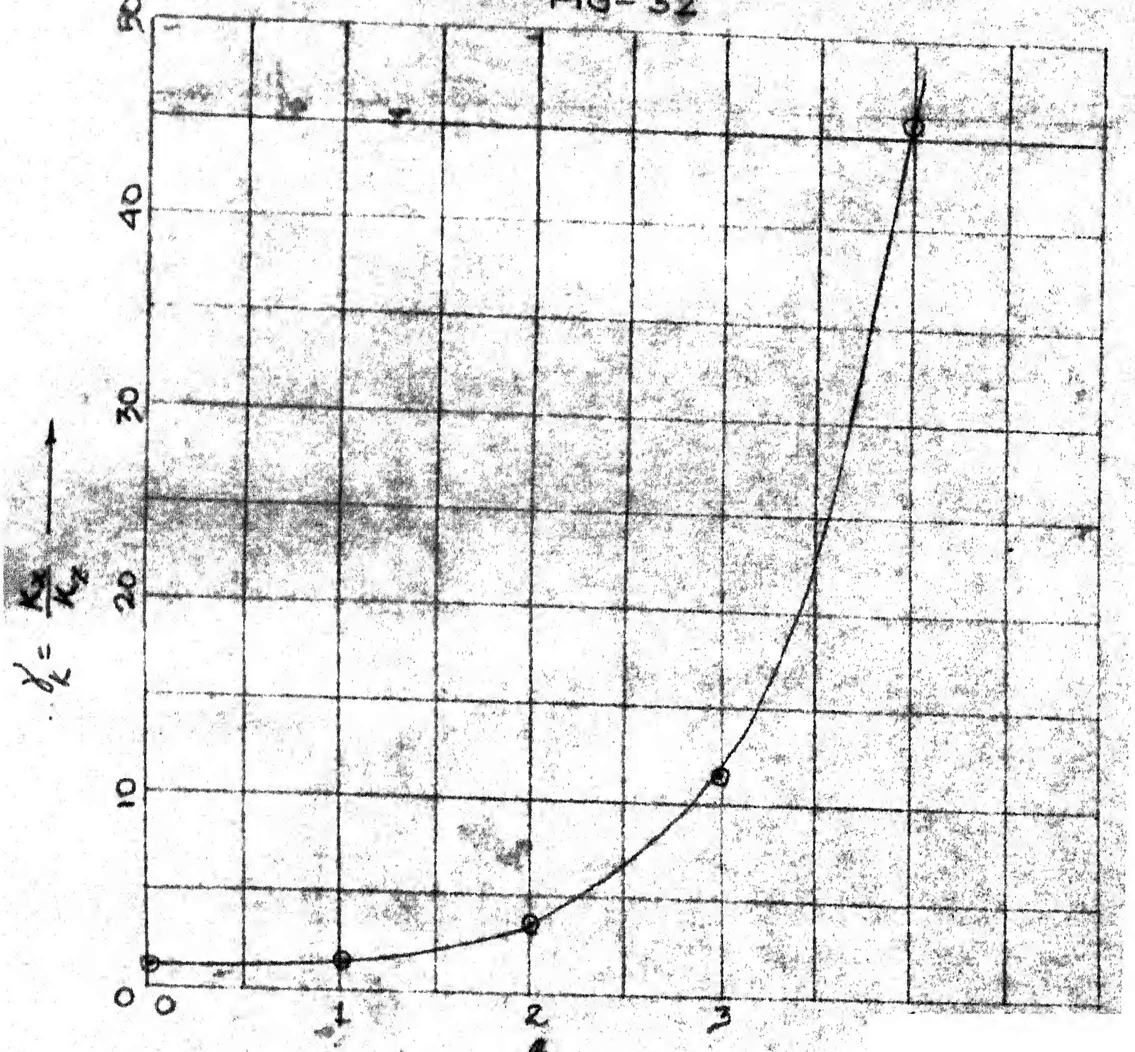
but $\alpha = \dfrac{t}{m}$

FIG. 31



VARIATION OF K_x & K_z WITH β.

FIG- 32



VARIATION OF $\frac{K_x}{K_z}$ WITH β

8.3 DISCUSSIONS AND CONCLUSIONS:

The derived equation show that for small values of β that is when e_{av} is nearly equal to e_o i.e. when void ratio decreases at a very small rate with depth, K_x and K_z are approximately equal and γ_K is slightly more than 1. But when β has a high value i.e. when the average void ratio is very small compared to the void ratio at h = o, K_x is much larger than K_z and consequently, the value of permeability ratio γ_K is appreciably higher.

Graphical plots of $\frac{K_x}{\gamma}$ and $\frac{K_z}{\gamma}$ for different values of are shown in fig. 31. It is seen that for a particular soil (i.e. with a constant e_o and m) as 'e_{av}' decreases K_z decreases at faster rate than K_x. A plot of β vs. γ_K is shown is fig.32. As seen from the figure for values of β within 0 to 1 γ_K remains almost equal to 1 and increases at a rapid rate with increasing β. At large β values i.e. with low e_{av} as compared to e_o, the permeability ratio is very large indicating a predominantly horizontal flow. Another interesting point to note is that equation 8.10, clearly shows that the permeability ratio is a function of void ratios e_c, e_{av} and the rate of change of log **K** with e (i.e. m) and is independent of actual permeability values in the soil strata.

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CHAPTER 9

CONCLUSIONS & SCOPE FOR FURTHER STUDY

9.1 CONCLUSIONS

The following conclusions can be drawn from the present investigations.

1. The radial permeameter designed & developed and as reported in chapter 3 is ideally suitable for the determination of radial coefficient of permeability K_r. The instrument can be used both as constant head and variable head permeameter. The expressious for K_r for both constant and variable head as derived and shown in chapter 3 is very compact in form & easy to use.
2. All the radial and vertical permeability tests conducted have shown that kaolinite and kaolinite with dispersing and flocculating agent have a higher radial permeability coefficient than vertical permeability coefficient. This is due to the flaky shape & orientation of individual particles with respect to the horizontal and vertical axes.
3. The limited experimental results have shown that for the samples tested, permeability ratio ($r_K = K_r/K_V$) is inversely proportional to the void ratio and it lies between 1 to 1.6 within the void ratio range of 0.655 to 0.85 depending

upon the percentage of dispersing or flocculating agent mixed with the kaolinite or in other words depending upon the degree of par allelism.

4. At the same void ratio, kaolinite with 5% dispersing agent have shown more radial permeability and permeability ratio than kaolinite with 5% flocculating agent essentially due to more degree of parallelism in first case than second.

5. The hypothesis for structural scale of soils based on permeability ratio, as forwarded in chapter 6 gives an opportunity to assign an unique value to any soil within the structural scale.

6. A study of shape factor for kalpi and ganges sand as reported in chapter 7 reveals that

   a. Shape factor 'C' is a function of void ratio and grain size.

   b. For larger grain size shape factor C assumes an unique value irrespective of void ratios.

   c. The rate of change of shape factor with void ratio is very less for large grain size.

   d. For ganges sand, shape factor increases and then decreases after a certain void ratio but for kalpi sand the decreasing tendency is absent within the void ratio range tested.

7. Expressions for horizontal & vertical permeability and permeability ratio for a case where void ratio linearly decreases with depth have been analytically derived and shown in chapther 8. The derived equations show that

   a. When void ratio decreases at a very small rate with depth, K_x & K_z are approximately equal and r_K is slightly more than one.

   b. When void ratio decreases at a very fast rate with depth, K_x is much larger than K_z and consequently the permeability ratio is very high.

   c. For a particular soil as average void ratio e_{av} over a depth decreases K_z decreases at a faster rate than K_x.

   d. Permeability ratio is a function of void ratios e_o, e_{av} and the rate of change of log K with e and is independent of actual permeability values in the soil strata.

.2 SCOPE FOR FURTHER STUDY

The following are a few areas where further research would be a benifit to engineers in understanding the aniosotropic permeability behaviour of soils.

1. Radial permeability and permeability ratio characteristics for a layered system of soils. The effect of inclined layers may also be investigated.

2. Effect of particle orientation and shape of Montmorillonite & Illite on the aniosotropic permeability behaviour is warranted.
3. Further work is necessary to verify the validity of hypothesis for structural scale, forwarded here, for several types of soils and over a larger range of void ratios.
4. An investigation for shape factor other than kalpi and ranges sand is necessary to study and generalise the factors on which shape factor depends.
5. Analytical investigations for depth dependent aniso-tropic permeability for different distributions of void ratio with depth will be of much use to the field engineers.

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REFERENCES CITED IN THE THESIS

1. Don Kirkham, "Measurement of the hydraulic conductivity of soil in place" - Symposium on permeability of soils, A.S.T.M., StP 163 1954.

2. T.Talsma, "Measurement of soils anisotropy with piezometers" -Journal of soil science - 1960 - 61, vol. 11 - 12 , p. 159.

3. D.W. Taylor, "Fundamentals of Soil Mechanics - page 118.

4. Evans K.E. ' ´ , "A note on the average coefficients of permeability for a stratified soil mass" - Geotechnique, Vol. 12, P. 145 - 146, 1962.

5. T. William Lambe, "The permeability of fine grained soils" - "Symposium on permeability of soils - A.S.T.M.Sept. No. 163, June 15, 1954.

6. Kozeny, J.S., Sitzungsber, Akad. Wissenschaftess, vienna: 1927.

7. Charles I. Mansur and Rudy J. Dietrich - "Pumping tests to determine permeability ratio" - Journal of soil mechanics and foundation engineering division - A.S.C.E. July, 1965, page 151.

8. Bernt Jackobson - "Isotropy of clays" - Geotechnique, 1955, Vol. 5 , P. 23.

9. T.C. Kenney - "Permeability ratio of repeatedly layered soils"- Geotechnique, Dec. 1963, No. 4, P.325.

10. K.A. Siraskar and V.J. Patel - "Equivalent permeability of nonhomogenous soils" - Indian National Society of Soil Mechanics Foundation Engineering April 1967, P. 153.

11. H. Darcy, "Les fontaines publiques de la ville de Dijon", Paris: Dijon 1856.

12. Report of the Subcommittee on permeability and infiltration, committee on Terminology, Soil Science Society of America - L.A. Richards, Chairman - Proc. Soil Sci. Amer. Vol 16, 1952, P.85.

13. M.Anandakrishnan and G.H. Varadara julu - "Laminar and Turbulant flow of water through sand" - Journal of Soil Mechanics and Foundation Division A.S.C.E., September, 1963, Vol. 89, No. S.M. 5, P. 1.

14. P.YA. Polubarinova - Kochina, "The theory of Ground Water Movement" State press. Moscow, 1952.

15. Swartzendruber, D. - "Modification of Darcy's law for the flow of water in soils" - Soil Science Vol. 93, 1962 P. 22 - 29.

16. V.S. Florin, "Compression of the Earth's substance, and Filtration as the presence of variable porosity, Taking into consideration the effects of combined water" - izv. Akad. Nank, SSSR, otdel. Tekh, Nank 11, 1625, 1951.

17. M.K. Hubbert, "The theory of ground water motions" - J.Geol. 48: 785 - 944.

18. Leonard J. Langfelder, C.F. Chen, and John A. Justice, "Air permeability of compacted cohesive soils" - Journal of S.M.F.E. - Proc. of A.S.C.E. Vol. 94. No SN 4, July 1968, pp 981 - 1002.

19. J.Kozeny, Berichte Wien Akademie, Vol. 136 a, P. 271 (1927).

20. P.C. Carman, "Fluid Flow Through Granular Beds", - Transactions, Inst. Chemical Engrs. (London), Vol. 15, P. 150 (1937).

21. A.G. Londar, "The computation of permeability from simple soil tests" - Geotechnique, Vol. 3, 1952 P. 165.

22. R.W. Smith & W.O. Stallman, "Measurement of permeabilities in ground water investigations" - Symp.. on perm. of soils, 1954, ASTM, StP no.163 P. 80 - 97.

23. E.C. Childs, N. Colligeorge & J.W. Holmes, "Permeability measurements in the field as an assessment of aniosotropy and structure development" - Journal of Soil Science, 1957, P. 27 - 41.

24. E.C. Child, "The aniosotropic hydraulic conductivity of soil"- Journal of Soil Science, 1957, Vol. 8, P 42 - 47.

25. Charles I. Mansur and Rudy J. Dietrich, "Pumping test to determine permeability ratio",- Journal of S.M.F.E. Divn. A.S.C.E., July 1965 P. 15.

26. Charles I. Mansur, "Laboratory and in situ permeability of sand " - Journal of S.M.F.E. Divn. A.S.C.E. 1957, Jan. paper No. 1142.

27. "Final Report, Soil Solidification Research, Vol.2, Fundamental properties, clay water systems", Cornell University, Ithaca, N.Y. Sept. 1951.

28. A.S. Michaels and C.S.Lin, "The permeability of kaolinite", Industrial and Engineering Chemistry, Vol. 46, June 1954, P. 1239 - 1246.

29. T. William Lambe, "The structure of Inorganic Soil", Proceedings, A.S.C.E., Vol. 79 pp. 315 -49, October 1953.

30. A.S. Michaels and T.William Lambe, "Laboratory evaluation of polyelectrolytes as Soil Flocculants and Aggregate stabilizers" - Agricultural and Food chemistry, Sept 1953, p.p. 835 - 843.

31. Muskat M., "The flow of homogeneous fluids through porous media" - Newyork; McGiraw Hill, 1937.

32. R.F. Scott, "Principles of Soil Mechanics" - Addison Wesley Publishing Company.

33. Philip Norton Buchanon, "Effect of temperature and adsorbed water on permeability and consolidation characteristics of sodium and calcium Montmorillonite" - Ph.D. Thesis, August 1964, Texas, A&M University.

34. Miss Margaret D. Foster, "Effect of Swelling on permeability"- A discussion on ref. 33.

35. G.T. Orlob and G.N. Radhakrishna, "The effect of entrapped gases on the hydraulic characteristics of porous media"- American Geophysical Union, Vol. 39, 1958, p. 648.

36. Arthur F. Pillisbury and David Appkman, "Factors in permeability changes of soils and inert granular materials" - Journal of Soil Science, Vol. 59, 1945, P.115.

37. L.E. Allison, "Effect of micro - organism on permeability of soil under prolonged submergence" - Soil Science, Vol.63, 1947, P. 439.

38. Sherard, Woodward, Gizienski and Clevenger, "Earth and Earth-Roch Dams" - John Wiley and Sons, Inc. Newyork.

39. A.E. Seheidegger, "The physies of Flow Through porous media"- OxFord Univ. Press, London.

40. D.Flaunery and Don Kirkham, "A soil core Water permeameter for field use" - Soil Science Vol. 97, 1964, P. 233.

REFERENCES FOR FURTHER STUDIES

1. Lutz, J.F.,& Kemper, W.D., "Intrinsic permeability of clay as affected by clay water interaction" - Soil Science, Vol. 88, P. 83 - 90, 1959.

2. Christiansen, T.E., "Effect of entrapped air upon the permeability of soils" - Soil Science, Vol. 58, P.353 - 365,1944.

3. C.W. Rose & A.Krishnan, "A method of determining hydraulic hydraulic conductivity characteristics for non swelling soils in situ" - Soil Science, Vo. 103, P. 369, 1967.

4. E.G. Young & G.D. Towner, "A study of some three dimensional field drainage problems" - Soil Science Vol. 95, P. 369. 1968.

5. E.C. Childs & E.G. Young, "A Study of some three dimensional field drainage problems" - Soil Science, Vol 92,P.15, 1961.

6. E.C.Childs & A. Ponlovassilis, "An oscillating permeameter"- Soil Science, Vol 90, P.326, 1960.

7. M.Fireman, "Permeability measurements on disturbed soil samples" - Soil Science, Vol 58, P. 337 - 353, 1944.

8. Fathi Amer, "Relation of laboratory hydraulic conductivity to texture, aggregation and soluble calcium plus magnesium percentage"- Soil Science, Vol 89, P.45, 1960.

9. T. Talsma & S.E. Flint, "Some factors determining the hydraulic conductivity of subsoil with special reference to tile drainage problems" - Soil Science, Vol. 85, P. 198, 1958.

10. V.S. Aronovici, "The mechanical analysis as an index of subsoil permeability" - Soil Science Soc. Amer. Proc. 11 , P P. 137 - 141, 1946.

11. L.D. Baver, "Soil permeability on relation to non capillary porosity" - Soil Science Soc. Amer. Proc. 3, P.52 - 56, 1938.

12. E.C. Childs & N. Collisgeorge, "The permeability of porous materials" - Proc. Royal Soc. (Londan) A 201 : P. 392-405, 1950.

13. C.H. Diebold, "Permeability and intake rates of medium textured soils in relation to silt content & degree of compaction" Soil Science Soc. Amer. Proc. 18, P. 339, 1954.

14. A.M. O'Neal, "Soil characteristics significant in evaluating permeability" - Soil Science Vo. 67, P.403, 1949.

16. W.R. Gardner & R.H. Brooks, "A descriptive theory of leaching"- Soil Science, Vo. 83, P. 295, 1957.

17. W.D. Kemper and T.F. Lutz, "Hydraulic conductivity in large channels as determined by an electric analog" - Soil Science, Vol. 81, P. 283, 1956.

18. M. Maasland, "Measurement of Hydraulic conductivity by the auger hole method in Aniosotropic soil" - Soil Science, Vol.81, P.379, 1956.

19. J.N. Luthin & Don Kirkham, "A piezometer method for measuring permeability of soil in situ below a water table" - Soil Science Vol. 68, P.349, 1949.

20. D. Swartzendruber, "Water movement into soil from idealized vertical Mulch channels"- Proc. Soil Science, Vol.24, P.152,1960.

21. Gupta & Swartzendruber, "Entrapped air content and hydraulic conductivity of quartz sand during prolanged liquid flow" - Proc. Soil Sci. Vol. 28, P. 9, 1964.

22. Paul R. Day & George G. Holmgren, "Microscopic changes in soil structure during compression" - Proc. Soil Science, Vol. 16, P. 73, 1952.

23. J.K. Mitchell, Don, R. Hooper & R.G. Campanella, "Permeability of compacted clay" - J. of Sm. & Foundation Engineering division A.S.C.E. July 1965, P. 41.

24. "The permeability of soils & the concept of stationary boundary layer", - Proc. Amer. Soc. for Testing & Materials, Vol 57, P. 1195, 1957.

25. R.J. Miller & P.F. Low, "Thresh hold gradient for water flow in clay systems" - Proc. Soil Science of Amer. Vol. 27, No. 6, P. 605 - 609, 1963.

26. T.W. Lambe, "The engineering behaviour of compacted clay"- A.S.C.E. Proc. May 1956, Paper No. 1655,

27. L. Bjerrum & J. Huder, "Measurement of the permeability of the permeability of compacted clays"- Proc. 4th International Conf. on Soil Mechanics and Foundation Engineering, Londan, Vol 1, P. 6 - 10, 1957.

28. J.K. Mitchell, "The fabric of natural clays & its relation to engineering properties" - Proc. H.R.B., Vol. 35, P.693 - 713, 1956.

29. D.G. Mekinley, "A laboratory study of rates of consolidation with particular reference to condition of radial pore water drainage" - Proc. 5th International Conf. on Soil Mechanics and Foundation Engineering, P. 225, 1961.

30. P.W. Rowe, "Measurement of the coefficient of consolidation of lacustrine clay" - Geotechnique Vol. 9, P. 107 - 118, 1959.

31. M.J. Hvorslav ," Physical components of shear strength of saturated clays" - A.S.C.E. Research Conf. on shear strength of cohesive soils, Boulder, Colorodo,P. 169 - 273, 1960.

32. Y. Arnimelech & Z Nevo, "Biological clogging of sand" - Soil Science, Vol. 8, P. 222, 1964.

33. Dan Zaslavsky, "Saturated and unsaturated flow equation in a unstable porous medium" - Soil Science, Vol. 8, P. 317, 1964.

34. W.P. Kelley, "Soil properties in relation to exchangable cations and kinds of exchange materials" - Soil Science, Vol. 8, P. 408, 1964.

35. E.J. Doering, "A direct method for measuring the upward flow from water table" - Soil Science, Vol. 96, P. 191, 1963.

36. E.G. Young , "An infiltration method of measuring the hydraulic conductivity of unsaturated porous media" - Soil Science, Vo. 9 P.307, 1964

37. Dan Zaslavsky, "Theory of unsaturated flow into a non uniform Soil Profile" - Soil Science, Vol. 97, P. 400, 1964.

38. L.D. Baver, "Soil Physics" - 2nd Edtion 1948.

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